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INTERACTIVE DESIGN OF CONTINUOUS FLEXURAL STEEL MEMBERS

by
Yi Jiang

A thesis submitted to
the Faculty of Graduate Studies and Research
in the partial fulfillment of the requirements

For the degree of
Master of Engineering

Department of Civil and Environmental Engineering
Carleton University, Ottawa
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"Interactive Design of Continuous Steel Members

Submitted by Yi Jiang

**in partial fulfilment of the requirements for the degree of
Master of Engineering**

Associate Professor S.J. Kennedy, Thesis Supervisor

**Professor J.L. Humar, Chair
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Abstract

The thesis presented, describes the theoretical background, purpose and examples of a graphics oriented, event driven program IDCSM (Interactive Design of Continuous Steel Members) for the analysis and design of continuous steel members with prismatic or non-prismatic cross sections.

The finite element developed for the analysis module combines the solutions of the governing differential equations for inplane flexural behaviour with the finite element method. Only one element is required to model each span of the continuous member (prismatic or otherwise). The principal advantages are simplicity, reduced computational effort and accuracy in the description of internal forces and in its displaced shape.

The design module accounts for the effect of various types of lateral supports (restraint to top or bottom flanges), multiple load combinations, variations in cross-section within each span and designs the continuous plate girder in accordance with CAN/CSA-S16.1-M89. Bearing stiffeners and intermediate stiffeners are designed automatically. IDCSM can be used to evaluate specified structures or to design structures automatically. The analytical results and designs produced by IDCSM have been verified with known solutions.

The graphic user interface for IDCSM provides a logical sequence of event driven windows that simulate the design process, clear scaled realistic images of the design problem as it is developed by the engineer (realistic boundary conditions and lateral bracing), a graphical output which displays internal force distributions, deflected

shapes and the designed plate girder.

IDCSM also provides different levels of reporting (summaries or detailed) which give intermediate calculations, design criterion and design strategies. As part of this thesis the algorithms, solution technique and program module were developed to predict the ultimate load capacity of eccentrically loaded bolt and weld groups for any orientation of load and for any orthogonal arrangement of bolts or welds. IDCSM is programmed in C language and integrated within a Hoops graphics user interface package.

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List of Symbols

A	area
A_k	weighting factor which is derived from Legendre polynomials
A_m	effective size $\times$ length of weld for fillet welds
$[A_v]_i$	end reaction of i th segment due to unit load applied to the base beam
A_w	web area
a	distance from the left end of element
B_r	factored bearing resistance of a member
b	depth of section; width of a section flange
D	displacement
d	width of fillet weld
$\{d\}$	nodal displacement vector
$d\theta$	relative angle of rotation between two cross section
E	elastic modulus of steel, 200 000 MPa
e	base of natural logarithms; distance between the center of gravity and the applied load
F_s	ultimate shear stress
F_y	yield strength, Mpa
$\{F\}$	nodal force vector
f_s	actual shear stress in the panel
h	optimum web depth, mm
I	moment of inertia of a section

I_s	moment of inertia of stiffener(single or pair) about an axis in the plane of the web
j	load case number
$[K]$	stiffness matrix
k	distance from outer face of flange to web toe of web-to-flange fillet
l	element length;length of fillet weld
l_m	length of segment
M_f	factored bending moment KN-m
M_1	ending moment caused by applied loads
M_r	resisting moment of the section
M_r'	reduced factored moment resistance
M_v	bending moment caused by virtual force
M_y	yield moment
m	moment
N	length of the bearing through which the load is delivered
N_i	shape functions
N_s	number of segments
$[N]$	bending shape function matrix
$\{P\}$	applied nodal load vector
Q	moment of the flange area about the neutral axis of cross-section
q	shear flow
R	fastener load at any given deformation
R_i	resisting force of a weld element
$(R_i)_x$	horizontal component of resisting force of a weld element
$(R_i)_y$	vertical component of resisting force of a weld element
R_{ult}	ultimate load attainable by fastener
$R_{u\parallel}$	ultimate strength of a longitudinal fillet weld loaded in shear
R_{θ}	ultimate strength of a fillet weld loaded in shear at any angle of loading
r	radius of rotation for the element of weld which first reaches its ultimate deformation

r_g	distance from trial location of the instantaneous centre to the vertical length of weld
$r_{\max}$	distance from instantaneous center of rotation to furthest fastener
r_n	distance from instantaneous center of rotation to i th fastener
S	elastic section modulus
s	spacing of welds (c/c)
t	thickness of flange
U	strain energy stored in the beam element
$u(x)$	axial displacement
V_f	factored shear
V_r	factored shear resistance
W	work done by external forces
W_i	weight factor
w	vertical displacement of the beam; web thickness
w_i	displacement in z direction at node i
X_u	ultimate strength as given by electrode classification number
x	coordinate along beam axis
x_0, y_0	location of instantaneous center
x_i, y_i	x, y coordinate of i th fastener
α	ratio of $\frac{a}{l}$
β_1	coefficient of area
β_2	coefficient of moment inertia
Δ	the midspan deflection; shearing, bending, bearing deformation of fastener and local bearing deformation of the connection plates
$\Delta_{\max}$	maximum fastener deformation at the fastener which is farthest from the instantaneous center
δA_c	virtual actions
$\delta \sigma$	virtual stress
$\{\delta \sigma\}_j$	virtual stresses vector for the condition of $(\delta A_c)_j = 1$
ϵ	real strain

ϵ_x	longitudinal normal strain
$\mu\lambda$	regression coefficients
$\{\Phi\}$	applied non-nodal load vector which is a function of coordinate x and load density
ϕ	resistance factor(which is equal to 0.9 for base metal); angle between applied load and x -axis
ϕ_w	resistance factor(which is equal to 0.67 for weld metal)
Π	potential energy
σ_t	tension stress
σ_y	yield stress
θ	rotation of the normal to beam axis
θ_i	rotation at node i

Chapter 1

Introduction

1.1 General

Early versions of computer software for structural analysis and design such as SAP77, STRUDL and NEWSAS, were only focused at analyzing structures by using the finite element method. This type of software provided structural engineers a powerful tool for calculating the internal forces and deformations of structures. The software lacked user interfaces and required hand calculations to prepare input data and for the design and drawing preparation. Subsequently, some functional oriented programs for design of structural elements(columns, beams) were developed subsequently. Generally, these were restricted to simple well designed problems, were not designer-friendly, and without a user interface system (mouse-driven menu, fully graphic input and output screen). The software was cumbersome and inconvenient for designers to use.

In 1984, a group of workstation users at Massachusetts Institute of Technology began the development of a user-friendly interactive interface called X window. The development of the innovative interface system in 1987 stimulated rapid development of graphic-oriented programs for interactive structural design. It is universally rec-

ognized that the development of designer-friendly computer software for interactive analysis and design of structures has been an important goal for structural engineering profession. The achievement of that goal depends on the development of comprehensive processes for evaluating loads, load effects and member resistances, the development of universal data bases, the development of object libraries and interfaces specific to structural engineering and the evolution of computer hardware and underlying software.

1.2 Analytical Models of Steel Members

The level of accuracy and sophistication the analytical model for structural analysis and design software is important, as it provides the most realistic mechanical response of the structure and its members subjected to the applied loads. From the results of the analysis, the design of the structure can be completed or the design of existing structures checked during automatic design procedures. Any change of geometric and material properties for the structural elements for or to the applied loads during the automatic design and redesign procedure will lead to a re-analysis of the structural behaviour.

The finite element method is the most commonly used numerical method for structural analysis. Many useful finite element models have been proposed by the following researchers Yang (1973), Zienkiewicz (1977), Murray (1988), Cook (1989) and Fleming (1989). A number of these have been incorporated into computer software for the interactive design of structures. For example, the analysis model for the computer program IDFSM (He, 1993) was based on the beam finite element analysis of a plane frame program developed by Murray (1988). In general, the nodal displacements and internal forces obtained from classical finite element methods will be exact for problems with linear moment distributions along beams and frames. For most problems the distributions are not linear and can not be described by conventional interpolation functions of the nodal values. One way in which a more accurate distri-

bution can be obtain is to model beams and columns with large number of elements. For example, He(1993) used between 100 and 300 elements for a single span. This leads to increased computations and data and is considered to be inefficient. To use a conventional beam finite element model to analyze continuous nonprismatic steel members accurately would result a very large number of elements. Hence the development of a simple and sufficiently accurate finite element model for a continuous steel member (non-prismatic in the general case) for practical applications has been and still remains a prime objective of many researchers.

1.3 Computer Software for Interactive Design of Steel Structures

The 1990's have been and currently are an active period for the development of designer-oriented CAD systems for structural engineering. Computer software for interactive design of structures for beam and column design has been developed by ECOM Associate, USA (1991). Typically, commercial member design software is very basic (simple straight application of current material standards and specifications) and does not handle a large number of realistic problems which require a description of lateral support, shape of bending moment diagram, description of coping and other factors that affect the members behaviour and capacity. The oversimplification of design parameters leads to severe limitations of its application. Rahimian (1992) describes an advanced integrated beam design program proposed by the Engineering Software Company, which can only be applied to the design of single span members. It is not valid for continuous beams or for beams with a variety of load combinations. W. E. S., (1992), Waterloo Engineering Software, developed a design package for Structural Optimization Design & Analysis (SODA) to design steel members according to CAN/CSA - S16.1 - M84. The design of beams is limited to a basic strength check, serviceability limit(deflection) check, a single interpretation of lateral torsional

buckling capacity (very limited application) and requires hand calculations of input. The design is semi-automatic and can only be verified by hand calculations. There is limited reporting of capacities. Based on an effective integration of CAD data structure and its associated user-interface, Anumba and Watson (1992) developed an innovative CAD system which makes the CAD system more designer-oriented. The key factor in addressing the need for a designer-friendly system was the user interface of the CAD system which bridges the gap between system functions and the designer perception. Chuang and Adeli (1993) developed a design-independent CAD window system by using an object-oriented paradigm and widget environment. This CAD system includes graphic input, output and an interactive user interface window than can be easily linked to any CAD package. The application of this CAD system for designing non-prismatic members is not possible. He (1993) developed an effective graphic-oriented computer program IDFSM for Interactive Design of Flexural Steel Members. This software has the capacity to design steel beams subjected to flexural and torsional loads, and includes the effect of lateral support, the influence of interactive buckling on the lateral-torsional buckling capacity and the bearing stiffeners. Like most existing software, IDFSM does not provide the design of plate girders, non-prismatic steel members nor continuous steel members but was developed to allow for modification to include these type of members.

1.4 Objective and Scope

It is evident from the preceding discussion, that the development of comprehensive computer software for interactive design of steel structures should be based on; (1) an effective structural analysis model; (2) a well established design procedures; (3) the state of the art; (4) available design and (5) an effective integration of the data structure and its associated user-interface. The objective of this thesis is to develop practical computer software for the interactive design of non-prismatic continuous steel members called, Interactive Design of Continuous Steel Members(IDCSM). To

overcome the limitations of existing beam finite element models, an exact beam finite element for first-order elastic analysis, prismatic or non-prismatic, single span or multi-span steel members with any prescribed boundary conditions under any combination of applied load will be developed. An effective interactive design module for the design of plate girders, non-prismatic steel members and continuous steel members according to CAN/CSA-S16.1-M84 will be developed. In conjunction, the corresponding graphical user interface will be developed to provide the proper environment for structural engineers to understand the behaviour of the structural member and the design produced by the program. The interface will give the user the ability to verify the design. The CAD system will satisfy the functionality requirements of ease of use and flexibility.

A complete analysis to determine the ultimate strength of eccentrically loaded bolted or welded connection for girder web splice plates in continuous members will be developed. It was beyond the scope of this thesis to include this analysis module and the corresponding design module for girder web splice plate connections.

Chapter 2

Finite Element Analysis of Continuous Flexural Non-Prismatic Members

2.1 General

For design purposes, the analysis program to describe the behaviour of the structure should have the following two basic attributes: (1) the analysis should provide a description of deformations and internal forces throughout the structure with acceptable accuracy, and (2) it should be easy to apply with minimal input and calculations.

The behaviour of flexural members has been studied by many researchers, William W. and James M. G. (1990), Fleming J. F. (1989), Cook, R. D. (1989). Generally, there are two classical numerical approaches for analyzing frame structures, the direct stiffness method and the flexibility method. Most analysis programs make use of the direct stiffness method. The advantage of using the flexibility method is that it can be used to determine the reactions directly without predetermined nodal displacements. Its major shortcoming that makes it impractical is that the flexibility matrix is unsta-

ble as the redundants are not unique and vary with the selection of calculation models. Displacement finite element analysis is also used in the structural analysis of framed structures, due to its simplicity, generality and efficiency. If an ordinary displacement finite element method is used to determine the displacements, the deformed shape and internal force diagrams for continuous beams, then each span must be subdivided into a large number of elements, He(1993). This leads to a very large number of equations to be solved and becomes ineffective. As a result, an accurate finite element module has been developed which combines the fundamental solution of governing equations with finite element technique. The number of elements is equal to the number of spans in the continuous beams, for both prismatic and non-prismatic members. The element stiffness matrices are determined using flexibility method which is convenient and effective in the case of non-prismatic members. The displacements within the elements are determined by summing the displacements determined from interpolation function of nodal displacements and those calculated from a corrected displacement function. The displacement corrected function is derived from governing differential equation of beam subjected to applied loads applied within the span. The internal forces within the elements are obtained from statics.

The analysis program has incorporated into PGIRD (plate girder) along with specially developed graphic user interfaces.

2.2 Beam Finite Element for Prismatic Flexural Members

The following analysis is based on the classical Euler-Bernoulli theory which assumes that plane sections remain plane after bending. This assumption implies that the shear deformation is negligible, which is a valid assumption for plate girders since they are thin walled open sections with span to depth ratios that are generally well excess of those considered to give deep beam behaviour. It is also assumed that

the flexural members behave linear elastically and that the deflections (including rotations) are small compared to the member's dimensions.

2.2.1 Basic Theory of Elastic Beam

Figure 2.1: Cross Section

According to conventional beam theory, the axial displacement, $u(x)$, at any point located at a distance x from one end can be expressed by

$$u(x) = -z \frac{dw}{dx} \quad (2.1)$$

where,

z = distance from the neutral axis to any point on the cross-section
in the z -direction (z -axis- minor principal axis)

w = vertical displacement of the beam

The longitudinal normal strain ϵ_x , referred to henceforth as the axial strain, is given by,

$$\epsilon_x = \frac{du}{dx}$$

or

$$\epsilon_x = -z \frac{d^2w}{dx^2} \quad (2.2)$$

The linear elastic stress-strain relation is defined by Hooke's Law as

$$\sigma_x = E\epsilon_x$$

where, E is the modulus of elasticity of the steel beam.

The strain energy stored in the beam element is

$$U = \int_v \frac{1}{2} \sigma_x \epsilon_x dv = \int_v \frac{E}{2} \epsilon_x^2 dv \quad (2.3)$$

in which v is the volume of the beam element.

Substitution of Equation. 2.2 into Equation 2.3, gives

$$U = \frac{E}{2} \int_l \int_A \left\{ z^2 \left(\frac{d^2 w}{dx^2} \right)^2 \right\} dA dx \quad (2.4)$$

The inner integral is equal to the moment of inertia about the y axis, I , where,

$$I = \int_A z^2 dA$$

The strain energy stored in the beam element, U can be written as

$$U = \frac{EI}{2} \int_0^l \left(\frac{d^2 w}{dx^2} \right)^2 dx \quad (2.5)$$

The potential energy, Π , of the beam element is given by Equation 2.6,

$$\Pi = U - W \quad (2.6)$$

where, W is defined as the work done by external forces.

According to the principle of stationary potential energy

$$\delta \Pi = 0 \quad (2.7)$$

Substituting Equation 2.6 into Equation 2.7, yields

$$\delta U = \delta W \quad (2.8)$$

where, δU can be obtained by differentiating Equation 2.5

$$\delta U = \int_0^l EI \left(\frac{d^2 w}{dx^2} \right) \left(\delta \frac{d^2 w}{dx^2} \right) dx \quad (2.9)$$

2.2.2 Element Displacement

The continuous flexural member is discretized into an assemblage of finite two node beam elements, each with the degrees of freedom for vertical displacement, w and rotation, θ at each node as shown in Figure 2.2. The vertical displacement for any point along the beam element can be approximated with a linear combination of cubic interpolation functions for each degree of freedom as

$$w = N_1 w_1 + N_2 \theta_1 + N_3 w_2 + N_4 \theta_2$$

or expressed in matrix notation as,

$$w = [N] \{d\} \quad (2.10)$$

where,

w_i = displacement in z direction at node i

θ_i = rotation at node i

and N_i are shape functions given by,

$$N_1 = 1 - 3\left(\frac{x}{l}\right)^2 + 2\left(\frac{x}{l}\right)^3$$

$$N_2 = x - 2\frac{x^2}{l} + \frac{x^3}{l^2}$$

$$N_3 = 3\left(\frac{x}{l}\right)^2 - 2\left(\frac{x}{l}\right)^3$$

$$N_4 = -\frac{x^2}{l} + \frac{x^3}{l^2}$$

or

$$[N] = \begin{bmatrix} N_1 & N_2 & N_3 & N_4 \end{bmatrix} \quad (2.11)$$

in which l is the element length, $[N]$ is the bending shape function matrix, and $\{d\}$ is the nodal displacement vector

$$\{d\} = \begin{bmatrix} w_1 & \theta_1 & w_2 & \theta_2 \end{bmatrix}^T$$

Figure 2.2: Beam Element

The first and second derivatives of the displacements, required for strain calculations, can be written as follows:

$$\frac{dw}{dx} = \frac{d[N]}{dx} \{d\} = [B] \{d\} \quad (2.12)$$

$$\frac{d^2w}{dx^2} = \frac{d[B]}{dx} \{d\} = [C] \{d\} \quad (2.13)$$

where,

$$[B] = \frac{d[N]}{dx} = \begin{bmatrix} B_1 & B_2 & B_3 & B_4 \end{bmatrix} \quad (2.14)$$

$$[C] = \frac{d^2[N]}{dx^2} = \begin{bmatrix} C_1 & C_2 & C_3 & C_4 \end{bmatrix} \quad (2.15)$$

and

$$\begin{Bmatrix} B_1 \\ B_2 \\ B_3 \\ B_4 \end{Bmatrix} = \begin{Bmatrix} \frac{dN_1}{dx} \\ \frac{dN_2}{dx} \\ \frac{dN_3}{dx} \\ \frac{dN_4}{dx} \end{Bmatrix} = \begin{Bmatrix} \frac{6x^2}{l^3} - \frac{6x}{l^2} \\ 1 - \frac{4x}{l} + \frac{3x^2}{l^2} \\ \frac{6x}{l^2} - \frac{6x^2}{l^3} \\ -\frac{2x}{l} + \frac{3x^2}{l^2} \end{Bmatrix}$$

$$\begin{Bmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \end{Bmatrix} = \begin{Bmatrix} \frac{dN_1}{dx} \\ \frac{dN_2}{dx} \\ \frac{dN_3}{dx} \\ \frac{dN_4}{dx} \end{Bmatrix} = \begin{Bmatrix} \frac{12x}{l^3} - \frac{6}{l^2} \\ -\frac{4}{l} + \frac{6x}{l^2} \\ \frac{6}{l^2} - \frac{12x}{l^3} \\ -\frac{2}{l} + \frac{6x}{l^2} \end{Bmatrix}$$

2.2.3 Element Strains, Stresses and Stiffness

Substituting Equation 2.13 into Equation 2.2 gives equation 2.16 which expresses the strain at any point in the element as a function of nodal displacements and the bending shape functions

$$\epsilon_x = -z[C]\{d\} \quad (2.16)$$

The derivatives of Equations 2.10, 2.12 and 2.13 are

$$\delta w = [N]\{\delta d\} \quad (2.17)$$

$$\delta \frac{dw}{dx} = [B]\{\delta d\} \quad (2.18)$$

$$\delta \left(\frac{d^2 w}{dx^2} \right) = [C]\{\delta d\} \quad (2.19)$$

Substituting Equation 2.13 and 2.19 into Equation 2.9, gives

$$\delta U = \{\delta d\}^T \left(\int_0^l EI[C]^T[C]dx \right) \{d\} \quad (2.20)$$

The equilibrium equations are obtained by substituting Equation 2.20 and the expression for δW which is equal to $\{\delta d\}^T \{F\}$ into Equation 2.8, and are expressed by,

$$\{F\} = [K]\{d\} \quad (2.21)$$

where, $\{F\}$ is the nodal force vector and $[K]$ is the stiffness matrix,

$$[K] = \int_0^l EI[C]^T[C]dx$$

which is defined explicitly as,

$$[K] = \frac{EI}{l^3} \begin{bmatrix} 12 & 6l & -12 & 6l \\ 6l & 4l^2 & -6l & 2l^2 \\ -12 & -6l & 12 & -6l \\ 6l & 2l^2 & -6l & 4l^2 \end{bmatrix} \quad (2.22)$$

2.2.4 Equivalent Nodal Load Vector

Applied loads can be classified as either nodal loads which act on nodes or non-nodal loads which act along the elements. The first can be applied directly in Equation 2.21 whereas the non-nodal loads must be changed into equivalent nodal loads first, and then applied.

The work done by the equivalent nodal loads can be expressed by

$$W_1 = \{d\}^T \{F\} \quad (2.23)$$

and the work done by applied loads can be expressed by

$$W_2 = \{d\}^T \{P\} + \int_0^l \{w\} \{\Phi\} dx \quad (2.24)$$

where,

$\{F\}$ = equivalent nodal load vector

$\{P\}$ = applied nodal vector

$\{\Phi\}$ = applied non-nodal load vector which is a function of
coordinate x and load density

Substituting Equation 2.10 into Equation 2.24, gives

$$W_2 = \{d\}^T \{P\} + \int_0^l \{d\}^T [N]^T \{\Phi\} dx \quad (2.25)$$

The virtual work done by equivalent nodal loads is equal to the virtual work done by applied loads,

$$\delta W_1 = \delta W_2 \quad (2.26)$$

where,

$$\delta W_1 = \{\delta d\}^T \{F\} \quad (2.27)$$

$$\delta W_2 = \{\delta d\}^T \{P\} + \int_0^l \{\delta d\}^T [N]^T \{\Phi\} dx \quad (2.28)$$

Substituting Equations 2.27 and 2.28 into Equation 2.26 and rearranging the expression in terms of the equivalent nodal load vector gives

$$\{F\} = \{P\} + \int_0^l [N] \{\Phi\} dx \quad (2.29)$$

The equivalent nodal load vector $\{f\}$ due to non-nodal loads is

$$\begin{aligned} \{f\} &= [f_1 \ f_2 \ f_3 \ f_4]^T \\ &= \int_0^l [N]^T \{\Phi\} dx \end{aligned} \quad (2.30)$$

in which f_1, f_2, f_3, f_4 are the equivalent vertical nodal loads and nodal moment at node 1, and the equivalent vertical nodal loads and nodal moment at node 2, respectively.

The equivalent nodal loads due to non-nodal concentrated force shown in Figure 2.3(a) can be determined from the following expression,

$$\{f\} = \sum_{i=1}^n [N]_i^T \{P\}_i$$

which expressed in full becomes,

$$\begin{aligned} \begin{Bmatrix} f_1 \\ f_2 \\ f_3 \\ f_4 \end{Bmatrix} &= \sum_{i=1}^n \begin{Bmatrix} (N_1)_i P_i \\ (N_2)_i P_i \\ (N_3)_i P_i \\ (N_4)_i P_i \end{Bmatrix} \\ &= \sum_{i=1}^n \begin{Bmatrix} (1 - 3\frac{x_i^2}{l^2} + 2\frac{x_i^3}{l^3}) P_i \\ (x_i - 2\frac{x_i^2}{l} + \frac{x_i^3}{l^2}) P_i \\ (3\frac{x_i^2}{l^2} - 2\frac{x_i^3}{l^3}) P_i \\ (-\frac{x_i^2}{l} + \frac{x_i^3}{l^2}) P_i \end{Bmatrix} \end{aligned} \quad (2.31)$$

where, $[N]_i$ is the value of shape function $[N]$ at the point of application of $\{P\}_i$, point i , x_i is the distance from left end of the member to the point of application of the concentrated load.

Figure 2.3: Load Case

The applied distributed load for the uniformly distributed load, trapezoidal shaped distributed load and triangular shaped distributed load cases are shown in Figure 2.3 (b), (c), and (d) respectively, and can be expressed for load case (b) as:

$$\{\Phi\} = q \quad (2.32)$$

and load case (c) as:

$$\{\Phi\} = \frac{q_1 - q_2}{c - b}(x - c) + q_1 \quad (2.33)$$

and load case (d) as:

$$\{\Phi\} = \frac{q}{d}(x - c) \quad (2.34)$$

Substituting Equation 2.11 into Equation 2.30, yields the equivalent nodal load functions for distributed loads.

$$f_1 = \int_c^{c+d} \left(1 - 3\frac{x_i^2}{l^2} + 2\frac{x_i^3}{l^3}\right) \{\Phi\} dx \quad (2.35)$$

$$f_2 = \int_c^{c+d} \left(x - 2\frac{x_i^2}{l} + \frac{x_i^3}{l^2} \right) \{\Phi\} dx \quad (2.36)$$

$$f_3 = \int_c^{c+d} \left(3\frac{x_i^2}{l^2} - 2\frac{x_i^3}{l^3} \right) \{\Phi\} dx \quad (2.37)$$

$$f_4 = \int_c^{c+d} \left(-\frac{x_i^2}{l} + \frac{x_i^3}{l^2} \right) \{\Phi\} dx \quad (2.38)$$

The equivalent nodal loads can be determined for load cases (b), (c) and (d) by substituting the corresponding function for $\{\Phi\}$ from Equations 2.32, 2.33 and 2.34 into Equations 2.35, 2.36, 2.37 and 2.38, respectively.

2.3 An Accurate Beam Element for Non-Prismatic Flexural Members

To minimize the calculations for the analysis of continuous flexural members with non-prismatic cross-sections, an accurate beam finite element based on the fundamental solution for flexure and the displacement finite element has been developed. Each span of the member is considered as an element, and each element consists of several segments with their own section property. The element stiffness matrix is derived by using flexibility method, in which the conventional unit-load technique is employed to determine the flexibilities of each element. Only inplane bending is considered. The displacements are assumed to be small.

2.3.1 Stiffness Matrix for Non-Prismatic Member

A typical non-prismatic member, shown in Figure 2.4(a), consists of three segments with different cross-sections. The stiffness and fixed-end reactions are a function of the geometric properties for each segment. To model this member using only one classical beam element would be difficult, however, it can easily be modelled using flexibility analysis.

The simply supported beam, shown in Figure 2.4(b) was used as the basis for calculating the flexibility of the element.

(a). beam element with three segments

(b). released element

Figure 2.4: Non-Prismatic Element

The terms indicated in Figure 2.4 are defined as,

α_1, α_2 = distance coefficient

l = length of element

β_1 = ratio of the area of a segment to the segment with the smallest segment area

β_2 = ratio of the moment of inertia of a segment to the segment with the smallest moment of inertia

A = the smallest cross section area in the three segments

I = the smallest moment of inertia in the three segments

The complementary virtual work done by external loads can be written as

$$\delta W^* = \{\delta A_c\}^T \{D\} \quad (2.39)$$

where,

δA_c = virtual actions

D = real displacement

The complementary virtual work done by internal forces can be written as

$$\delta U^* = \int_v \{\delta \sigma\}^T \{\epsilon\} dV \quad (2.40)$$

where,

$\delta \sigma$ = the virtual stress

ϵ = real strain

Applying the principle of complementary virtual work, yields

$$\delta W^* = \delta U^* \quad (2.41)$$

Substituting Equations 2.39 and 2.40 into Equation 2.41, gives

$$\{\delta A_c\}^T \{D\} = \int_v \{\delta \sigma\}^T \{\epsilon\} dV \quad (2.42)$$

For the case where a unit load $(\delta A_c)_j = 1$ and all other elements of $\{\delta A_c\}$ are equal to zero, Equation 2.42 becomes,

$$(1)D_j = \int_v \{\delta \sigma\}_j^T \{\epsilon\} dV \quad (2.43)$$

where, $\{\delta \sigma\}_j$ is virtual stresses vector for the condition where $(\delta A_c)_j = 1$.

For the slender members, like beams, the integration over the volume may be replaced by integration over the length of the virtual stress resultants and corresponding internal displacements. Considering that the beam is behaving primarily as a flexural

member, Equation 2.43 can be simplified as,

$$\begin{aligned} (1) D_j &= \sum_{i=1}^{N_s} \int_l M_v d\theta \\ &= \sum_{i=1}^{N_s} \int_l \frac{M_v M_l}{EI} dx \end{aligned} \quad (2.44)$$

where,

$d\theta$ = relative angle of rotation between two cross section

M_v = bending moment caused by the virtual force

M_l = bending moment caused by applied loads

N_s = number of segments

j = load case number

The displacement D_j due to one load case in Equation 2.44 can be written explicitly in terms of the moments at each end of the segment.

$$D = \frac{l_m}{6EI} (2m_l^l m_v^l + 2m_l^r m_v^r - m_v^l m_l^r - m_v^r m_l^l)$$

or

$$D = \begin{bmatrix} m_v^l & m_v^r \end{bmatrix} \begin{bmatrix} \frac{l_m}{3EI_m} & -\frac{l_m}{6EI_m} \\ -\frac{l_m}{6EI_m} & \frac{l_m}{3EI_m} \end{bmatrix} \begin{Bmatrix} m_l^l \\ m_l^r \end{Bmatrix} \quad (2.45)$$

The symbols given in Equation 2.45 are illustrated in Figure 2.5 and defined as,

l_m = the length of segment

m = moment

subscript v = virtual action

subscript l = load case

superscript r = right

superscript l = left

Figure 2.5: Beam Segment

The flexibility at the either end of the element is a function of the flexibility of each segment, and it is derived using matrix multiplication, as,

$$[F] = \sum_{i=1}^{N_s} [A_v]_i^T [F_m]_i [A_v]_i \quad (2.46)$$

where,

$[A_v]_i$ = the end reaction of ith segment due to unit load applied to the base beam

$$[A_v]_i = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 - \alpha_1 & -\alpha_1 \\ 0 & -(1 - \alpha_2) & \alpha_2 \end{bmatrix} \quad (2.47)$$

$[F_m]_i$ = the flexibility matrix of ith segment

$$[F_m]_i = \frac{(\alpha_2 - \alpha_1)l_m}{E} \begin{bmatrix} \frac{1}{\beta_1 A} & 0 & 0 \\ 0 & \frac{1}{3\beta_2 I} & -\frac{1}{6\beta_2 I} \\ 0 & -\frac{1}{6\beta_2 I} & \frac{1}{3\beta_2 I} \end{bmatrix} \quad (2.48)$$

The stiffness sub-matrix $[X]$ gives the reactions corresponding to unit joint displacements for the beam element and is obtained by inverting Equation 2.46.

$$[X] = [F]^{-1} = \begin{bmatrix} A_c & 0 & 0 \\ 0 & S_l & T \\ 0 & T & S_r \end{bmatrix} \quad (2.49)$$

Definitions for the terms in the matrix are:

A_c = reaction corresponding to an axial displacement (A_c is assume to be zero for the beam element).

$S_l(S_r)$ = left (right) joint reaction corresponding to left (right) joint rotation of the beam element.

T = left (right) joint reaction corresponding to right (left) joint rotation of the beam element.

Rearranging the terms, the stiffness matrix of non-prismatic beam element can be expressed as,

$$[S]_m = \begin{bmatrix} \frac{S_l+S_r+2T}{l^2} & \frac{S_l+T}{l} & -\frac{S_l+S_r+2T}{l^2} & \frac{S_r+T}{l} \\ \frac{S_l+T}{l} & S_l & -\frac{S_l+T}{l} & T \\ -\frac{S_l+S_r+2T}{l^2} & -\frac{S_l+T}{l} & \frac{S_l+S_r+2T}{l^2} & -\frac{S_r+T}{l} \\ \frac{S_r+T}{l} & T & -\frac{S_r+T}{l} & S_r \end{bmatrix} \quad (2.50)$$

2.3.2 Equivalent Nodal Load Vector for Non-Prismatic Members

As described in section 2.1, the nodal reactions depend on the flexibility characteristics of the corresponding segments of the beam element. The nodal load vector for non-prismatic elements can not be obtained by using the work-equivalent load vector method. To determine the equivalent nodal loads the work-equivalent load vector method must be combined with the flexibility method. This is carried out in two

steps. The first step is to determine the end-reactions for each segment using the work-equivalent load method described in preceding section. Second, the fixed-end reactions for each element due to the equivalent loads applied at the ends of each segment, as determined in the first step, are calculated using the flexibility method.

Figure 2.6 (a) illustrates the the equivalent loads obtained by using Equations 2.31 through 2.38, which are applied at the ends of each segment. The loads f_1 and f_2 applied on the left end of the element, and the loads $f_{2N,+1}$ and $f_{2N,+2}$ applied on the right end of the element are directly substituted into the appropriate location in the equivalent nodal load vector. The loads applied at both ends of each segment will be used to calculate the fixed-end reaction of the non-prismatic beam element shown in Figure 2.4 (b). The displacements at the end of the beam element, caused by the loads shown in Figure 2.6(b), can be expressed by

$$\{\delta_0\} = \sum_{i=1}^{N_s} [A_v]_i^T [F_m]_i \{A_l\}_i \quad (2.51)$$

in which, $[A_v]_i$ and $[F_m]_i$ are given by Equations 2.47 and 2.48, respectively.

The end moment vector in segment i $\{A_l\}_i$, due to all the loads within this element is in Figure 2.6(b).

In the case of simply supported beam element,

$$\{\delta_0\} = [\delta_0^1 \quad \delta_0^2]$$

where,

δ_0^1 = rotation at node 1

δ_0^2 = rotation at node 2

The flexibility equation can be expressed as,

$$[F] \{x_0\} + \{\delta_0\} = \{0\} \quad (2.52)$$

(a). equivalent load on segments

(b). end displacement

(c). end reaction

Figure 2.6: Equivalent Loads and End Reactions

where, $\{x_0\}$ in this particular case is given as,

$$\{x_0\} = [x_0^2 \quad x_0^4]^T$$

and,

x_0^2 = the fixed-end moment corresponding to the end rotation δ_0^1

x_0^4 = the fixed-end moment corresponding to the end rotation δ_0^2 (refer Figure 2.6).

Solving Equation 2.52, gives the fixed end moments in terms of the rotations,

$$\{x_0\} = -[F]^{-1} \{\delta_0\} \quad (2.53)$$

The other two end forces which correspond to w_1 and w_2 can be determined from

equations of equilibrium. The fixed-end reaction vector of a non-prismatic element can be expressed as,

$$\{f^f\} = [x_0^1 \ x_0^2 \ x_0^3 \ x_0^4]$$

The equivalent nodal equivalent load vector is given by the summation of the nodal load vector and the negative of the fixed-end reaction of the element.

$$\{F\} = \begin{Bmatrix} f_1 \\ f_2 \\ f_{2N_e+1} \\ f_{2N_e+2} \end{Bmatrix} - \begin{Bmatrix} x_0^1 \\ x_0^2 \\ x_0^3 \\ x_0^4 \end{Bmatrix} \quad (2.54)$$

2.4 Deflections and Internal Forces

Bending moment diagrams, shear force diagrams and deflected shapes (deflections along the length) are required for design.

In general, finite element analysis of frame structures will give exact solutions for internal force distributions and displaced shapes in frame structures if these are linear. If not, then the members must be discretized into a significant number of elements to give an accurate representation. The analysis detailed in section 2.2 and 2.3 uses one element to model each span for both prismatic and non-prismatic members. The displacement interpolation function given by Equation 2.10 is used to determine displacements at any point along the element. The use of the displacement interpolation functions alone may lead to unacceptable errors as they do not include the effects of the loads being applied in the region between the nodes. Based on the flexural equation of a beam subjected to applied loads, a displacement correction function can be introduced for each prismatic segment which considers the effects of non-nodal loads is added to the nodal displacements calculated from nodal loads to give the correct

displacements within each segment.

The governing differential equation of beam subjected to load $P(x)$ (flexure only) is given by,

$$EI \frac{d^4 w}{dx^4} = P(x) \quad (2.55)$$

The boundary conditions for the beam element shown in Figure 2.2 are,

$$\begin{cases} w = w_1 \\ \frac{dw}{dx} = \theta_1 \end{cases} \quad \text{for } x = 0 \quad (2.56)$$

$$\begin{cases} w = w_2 \\ \frac{dw}{dx} = \theta_2 \end{cases} \quad \text{for } x = l \quad (2.57)$$

The deflection w at any point along the beam element can be determined by solving Equation 2.55 for the boundary conditions and appropriate loading. For example in the case of a beam subjected to uniformly distributed load of intensity q , Equation 2.55 becomes,

$$EI \frac{d^4 w}{dx^4} = q \quad (2.58)$$

and the deflection w was determined as,

$$w = \frac{qx^2}{24EI}(x-l)^2 + [N] \{d\} \quad (2.59)$$

in which, x is the distance from left end to any point along the element. In general, the displacement w for any load case can be expressed as,

$$w = \Psi(x, P) + [N] \{d\} \quad (2.60)$$

where, $\Psi(x, P)$ is displacement correction function.

The displacement correction functions $\Psi(x, P)$ for each type of load is given as follows,

(1) Uniformly distributed load (applied along the entire span),

$$\Psi(x, P) = \frac{qx^2}{24EI}(x-l)^2 \quad (2.61)$$

(2) Concentrated load as shown in Fig. 2.3 (a). The displacement correction functions between points A and C are given as,

$$\Psi(x, P) = \frac{Pb^2x^2}{6EI} [3\alpha - (1 + 2\alpha)\xi] \quad (2.62)$$

and between points C and B as,

$$\Psi(x, P) = -\frac{Pa^2(l-x)^2}{6EI}[\alpha - (1+2\beta)\xi] \quad (2.63)$$

and at C as,

$$\Psi(x, P) = \frac{Pa^3b^3}{3EI^3} \quad (2.64)$$

where,

a, b, l, P = are distances, lengths and magnitude of the concentrated load defined in Figure 2.2(a)

$$\xi = \frac{x}{l}$$

$$\alpha = \frac{a}{l}$$

$$\beta = \frac{b}{l}$$

x = the distance from left end to the any point along the element where the displacement is required

(3) Uniformly distributed load as shown in Fig. 2.3(b). The displacement correction functions between points A and C are given as,

$$\Psi(x, P) = \frac{1}{6EI}(-R_A x^3 - 3M_A x^2) \quad (2.65)$$

and between points C and D as,

$$\Psi(x, P) = \frac{1}{6EI}(-R_A x^3 - 3M_A x^2) + \frac{q(x-d)^4}{4} \quad (2.66)$$

and between points D and B as,

$$\Psi(x, P) = \frac{1}{6EI}(-R_A x^3 - 3M_A x^2) + qc(x-a)^3 \frac{qc^3(x-a)}{4} \quad (2.67)$$

where,

a = the distance from the left end of element to the center of the distributed load

c, d, q = are distance and magnitude of loads as defined in Figure 2.3(b)

$$R_A = \frac{qc}{4l^3}(12b^2 - 8b^3 + c^2l - 2bc^2)$$

$$M_A = -\frac{qc}{12l^2}(12ab^2 + 3bc^2 - 2c^2l)$$

(4) **Triangular shaped distributed load** shown in Fig. 2.3(d). The displacement correction functions between points A and C are given as,

$$\Psi(x, P) = \frac{1}{6EI}(-R_A x^3 - 3M_A x^2) \quad (2.68)$$

and between points C and D as,

$$\Psi(x, P) = \frac{1}{6EI} \left[-R_A x^3 - 3M_A x^2 + \frac{q(x-d)^5}{20c} \right] \quad (2.69)$$

and between points D and B as,

$$\Psi(x, P) = \frac{1}{6EI} \left[-R_A x^3 - 3M_A x^2 + \frac{qc}{2}(x-a)^3 + \frac{qc^3}{12}(x-a + \frac{2c}{45}) \right] \quad (2.70)$$

where,

$$R_A = \frac{qc}{12l^3}(18b^2l - 12b^3 + c^2l - 2bc^2 - \frac{4c^3}{45})$$

$$M_A = -\frac{qc}{36l^2}(18ab^2 - 3bc^2 + c^2l - \frac{2c^3}{15})$$

(5) **Trapezoidal shaped distributed load** as shown in Fig. 2.3(c). The correction function is obtained by superposition of the uniformly distributed load and triangular shaped distributed load.

The deflection w within the element, as indicated by Equation 2.60, consists of two parts, the deflection $[N] \{d\}$ due to nodal loads (given by Equation 2.10) and the deflection $\Psi(x, P)$ due to non-nodal loads. The deflection w calculated from Equation 2.60 is exact.

The internal forces at any point within element can be determined using the principles of superposition and static equilibrium. The moment at any point along the beam element is equal to the sum of the moments about that location from nodal forces on the left end and all applied loads to the left of that location. The shear is equal to the sum of all forces including applied load and nodal load on the left of that location. For example, the bending moment M_x at point C in Figure 2.7 can be obtained from moment equilibrium of the isolated body of this member. M_x is the

sum of all moments about point C, which include the nodal moment M_A , the moment due to the nodal shear Q_A on the left end and the moments due to all loads applied the left of location C. In general, M_x can be expressed as,

$$M_x = M_A - Q_A x + M(x, P, q)$$

in which, $M(x, P, q)$ are moments due to non-nodal load about point C and are a function of the type of load, load location and load density, x , P , and q as illustrated in Figure 2.7.

Similarly the internal shear can be calculated as,

$$Q_x = Q_A + Q(P, q)$$

in which, Q_A is the nodal shear, $Q(P, q)$ is the shear due to non-nodal loads applied the left of point C.

Figure 2.7: General Free Body Diagram

2.5 Program Structure

A program module called PGIRD (plate girder) was developed for analyzing continuous flexural member with prismatic or non-prismatic sections and for designing the

continuous steel plate girder according to CAN/CSA S16.1-M89 and current fabricating practice in Canada. PGIRD is coded in C programming language and is linked to the main program by system calls from the main interactive application program IDCSM.

The input and output data are exchanged between PGIRD and IDCSM by file transmission. The output data used directly to plot the internal force diagram for each load combination as illustrated in Figure 6.2. The deflected shape for dead load and for each live load combination are plotted separately as illustrated in Figure 6.14 and 6.15. The values associated with the diagrams are listed in the detailed report described in Chapter 6.

2.6 Summary

An accurate finite element analysis was developed to give the deflected shape and internal force diagrams for continuous flexural non-prismatic members. The solutions of governing differential equation for flexure are combined with a finite element solution for beams to provide a beam element that is very sophisticated (prismatic or non-prismatic), convenient and easy to use and accurate.

The flexibility method was used to determine the stiffness matrix of any non-prismatic flexural member that can be modeled with three prismatic segments with different cross-sections. The displacement at any point along the length of the member is obtained by superposing the effects of both the nodal load displacements and the displacements due to non-nodal loads. The internal force distributions are determined from static equilibrium.

The two principal advantages of this proposed finite beam element over a conventional beam finite element are:

- (i) the element is simple and efficient. Only one element is required to model a prismatic or non-prismatic member, with any set of applied loads, spanning between

supports. Significant savings are achieved in computation time, and
(ii) the element provides exact values of displacements and internal forces at any
location.

Numerical examples are presented in Chapter 6.

Chapter 3

Design of Plate Girders

3.1 General

Steel plate girders are used as transfer girders and crane girders in steel buildings and are extensively used for composite bridges with spans less than 100m.

The Canadian standard CAN/CSA-S16.1-M89, Limit States Design for Steel Structures gives design provisions that are based on the current state-of-art, Basler, K. (1961), Basler and Thurliman (1961), Roberts (1981), Galambos (1988), and that have proven to be economical. The design philosophy leads to

- (1) a safe, reliable structure which performs the required function and
- (2) an economic structure in which the fabrication and maintenance costs are minimized.

A design algorithm in accordance with CAN/CSA-S16.1-M89 has been developed in conjunction with the finite element analysis program and a modified version of the graphic user-interface of IDFSM to proportion the plate girder for bending and shear; to calculate the location of lateral support; to prevent lateral torsional buckling; to design and locate the bearing stiffeners and stiffeners for shear; to calculate the capacity of both welded and bolted spliced connections for shear; and to determine the

weld details for the flange to web and stiffener to web connections.

3.2 Preliminary Sizing

The serviceability limit state for deflection often controls the sizing or proportioning of the cross-section of flexural members. The cross-section dimensions must be estimated with some accuracy at this stage to avoid unnecessary analysis design iterations as the relative stiffnesses, especially for non-prismatic and statically indeterminate structures, influence the deflections and internal force distributions.

A simplified formula was proposed by McGuire (1968), to give an estimate of the span to depth ratio (depth of section) by limiting the normal stress in bending at midspan, due to an equivalent uniformly distributed service load, to $0.6\sigma_y$. The derivation follows. The midspan deflection of simply supported beam with an equivalent uniform load, q applied along its length is given by,

$$\Delta = \frac{5}{384} \frac{ql^4}{EI} \quad (3.1)$$

The corresponding moment is a maximum and is equal to $ql^2/8$. The moment of inertia of the section, I is equal to $Sd/2$, where S is elastic section modulus, and d is the depth of the section. From beam theory, the extreme fiber normal stress due to bending at midspan, σ is equal to $ql^2/8S$. Substitute σ into Equation 3.1, to express the midspan deflection in terms of l/d ,

$$\Delta = \frac{80\sigma}{384E} \frac{l^2}{d} \quad (3.2)$$

The maximum allowable stress, based on AISC safety factor of 1.65, is $\sigma = 0.6\sigma_y$. Substitute this and $E = 200,000$ MPa into Equation 3.2 and rearrange the expression in terms of l/d to obtain Equation 3.3.

$$\frac{l}{d} = \frac{1584000}{\sigma_y} \frac{\Delta}{l} \quad (3.3)$$

where, Δ/l is a specified deflection limit.

For a given specified deflection limit Δ/l and yield stress σ_y , the span to depth ratio

can be estimated and typical values are summarized in Table 3.1. The preliminary value of d can be calculated.

Δ/l	$\sigma_y = 300MPa$	$\sigma_y = 350MPa$
1/240	22.0	18.9
1/300	17.6	15.1
1/360	14.7	12.6
1/600	8.8	7.5
1/800	6.6	5.7

Table 3.1: Estimate Value of l/d

3.3 Design for Bending and Shear

3.3.1 Preliminary Sizing of Flange and Web

The usual practice for selecting the section size is to first estimate the optimum web depth of the girder from an empirical expression which leads to economical designs and is expressed as a function of the maximum moment and the yield stress.

$$h \approx 540 \left(\frac{M_f}{F_y} \right)^{\frac{1}{3}} \quad (3.4)$$

where,

h = optimum web depth, mm

M_f = maximum factored moment kN-m

F_y = yield stress, MPa

The flange size is calculated on the assumption that the flange can reach full yield in compression without lateral-torsional buckling occurring (sufficient lateral supports

are provided). From moment equilibrium the area of compression flange can be expressed as,

$$A_f = bt = \frac{M_f \times 10^6}{F_y h} \quad (3.5)$$

where, A_f is the area of compression flange, or in terms of the flange thickness as,

$$t = \frac{M_f \times 10^6}{b F_y h} \quad (3.6)$$

where,

b = the width of flange

t = the thickness of flange

Plate girders are designed to meet class 3 requirements (noncompact sections, that can attain yield moment capacity), hence according to clause 11.1 of CAN/CSA-S16.2-M89 the width to thickness ratio of the flange is

$$\frac{b}{2t} \leq \frac{200}{\sqrt{F_y}} \quad (3.7)$$

Substituting Equation 3.7 into Equation 3.6 and rearranging the expression in terms of b ,

$$b = \sqrt{\frac{400 M_f \times 10^6}{F_y^{\frac{3}{2}} h}} \quad (3.8)$$

The flange thickness can then be calculated by Equation 3.6. Based on the application of the lower bound theorem, it is assumed that web carries all the shear due applied loads, hence the area of the web is given as

$$A_w = wh = \frac{V_f \times 10^3}{\phi F_s} \quad (3.9)$$

where,

w = width of the web, mm

V_f = factored shear force, kN

F_s = the ultimate shear strength (determined according to clause 13.4.1.1), MPa

ϕ = 0.9, resistance factor

The web thickness must lie between the following limits:

(i) minimum web thickness for corrosion, $w \geq 4.5$ mm

(ii) minimum web thickness based on the absolute maximum web slenderness, refer to clause 13.4.1.3, which prohibits local vertical buckling of the web due to the vertical component of the maximum compressive force in the flange.

$$h/w \leq 83000/F_y \quad (3.10)$$

(iii) maximum web thickness, based on a web slenderness that will not reduce the moment capacity of the section, refer to clause 15.4.

$$h/w = 1900/\sqrt{F_y} \quad (3.11)$$

The optimum web thickness should satisfy the shear requirements of clause 13.4.1.1 and give an approximate aspect ratio for the first panel of 1.25.

3.3.2 Flexural Resistance

The basic factored moment resistance for a class 3 flexural member is given by clause 13.5(b) as,

$$M_r = \phi M_y = \phi S F_y \quad (3.12)$$

where,

ϕ = 0.9, resistance factor

M_y = yield moment

S = elastic section modulus

For plate girders with a web slenderness, h/w greater than $1900/\sqrt{M_y/\phi S}$, the web does not meet class 3 requirements and will buckle prior to the extreme fibres of the web reaching yield due to bending (linear normal stress distribution). The moment

capacity is reduced and is given as,

$$M_r' = M_r \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{w} - \frac{1900}{\sqrt{\frac{M_f}{\phi S}}} \right) \right] \leq \phi M_v \quad (3.13)$$

where,

M_r' = reduced factored moment resistance

3.3.3 Shear Resistance

Based on the distribution of shear stress, the standard CAN/ CSA-S16.1-M89 assumes that the web alone resists the shear and that its resistance is expressed in clause 13.4.1.1 as,

$$V_r = \phi A_w F_s \quad (3.14)$$

where,

V_r = shear resistance

F_s = ultimate shear stress defined by clause 13.4.1.1 (a), (b), (c) and (d).

The ultimate shear stress of unstiffened girder web is different from that of stiffened girder web. For elastic buckling the critical shear stress is given as,

$$F_s = k_v \frac{\pi^2 E}{12(1 - \nu^2)(h/w)^2} \quad (3.15)$$

where, ν is Poisson's ratio and is taken as 0.3, $E = 200,000$ MPa and the shear buckling coefficient k , (Galambos 1988), for $a/h \geq 1.0$, and simply supported edges is given as,

$$k_v = 5.34 + \frac{4.0}{(a/h)^2} \quad (3.16)$$

and for fixed edges as,

$$k_v = 8.98 + \frac{5.6}{(a/h)^2} \quad (3.17)$$

in which, a/h is the aspect ratio of the webs.

In the case where the aspect ratio (a/h) is equal to infinity and for simply supported edges Equation 3.15 can be simplified as,

$$F_s = \frac{961200}{(h/w)^2} \quad (3.18)$$

Elastic shear buckling is assumed to be valid up to $0.8 \tau_y$, where, τ_y is the shear yield stress. Substituting this into Equation 3.18 gives the limit for elastic behaviour,

$$0.8\tau_y = \frac{0.8F_y}{\sqrt{3}} = \frac{961200}{(h/w)^2} \quad (3.19)$$

which can be expressed as,

$$\frac{h}{w} = 624 \sqrt{\frac{k_v}{F_y}} \quad (3.20)$$

where, k_v = shear buckling coefficient. Clause 13.4.1.1 gives the coefficient for this limit as 621. The limit between inelastic shear buckling and shear yielding, where $F_s = 0.66F_y$ (accounts for some strain hardening of the web) is,

$$\frac{h}{w} = 439 \sqrt{\frac{k_v}{(F_y)}} \quad (3.21)$$

The inelastic shear buckling strength is described by a straight interaction line between the two limits as is expressed as,

$$F_s = 290 \frac{\sqrt{k_v F_y}}{h/w} \quad (3.22)$$

The complete curve is illustrated in Figure 3.1.

Figure 3.1: Web Shear Strength - Unstiffened Web

The general formula for the ultimate shear strength of stiffened steel girder was derived from research by Basler and Thurlimann(1961) and includes the additional post buckling shear strength due to tension field action. For the elastic shear buckling region, $\frac{h}{w} \geq 621\sqrt{\frac{k_v}{F_y}}$, the expressions, composed of two terms, the elastic shear buckling strength and the tension field component, and is given as,

$$F_s = \frac{180760k_v}{(h/w)^2} + \frac{0.5F_y}{\sqrt{1 + (a/h)^2}} - \frac{156544k_v}{(h/w)^2\sqrt{1 + (a/h)^2}} \quad (3.23)$$

in which k_v is shear buckling coefficient and is equal to

$$k_v = 5.34 + \frac{4.0}{(a/h)^2} \quad (3.24)$$

(for $a/h \geq 1$) and

$$k_v = 4.0 + \frac{5.34}{(a/h)^2} \quad (3.25)$$

(for $a/h < 1$)

The coefficients 180760 and 156544 are simplified to 180,000 and 156,500 respectively,

in CAN/CSA-S16.1-M89. The maximum web slenderness of $83000/F_y$ still applies. The maximum ultimate shear strength and slenderness limit is the same as that for unstiffened girder web. For inelastic shear buckling, additional tension field capacity is only applicable for web slenderness greater than $502\sqrt{k_v/F_y}$ and is expressed by the following transition curve between the limits $502\sqrt{k_v/F_y}$ and $621\sqrt{k_v/F_y}$ as,

$$F_s = 290 \frac{\sqrt{k_v F_y}}{h/w} + \frac{0.50 F_y}{\sqrt{1 + (a/h)^2}} - \frac{251 \sqrt{k_v F_y}}{h/w \sqrt{1 + (a/h)^2}} \quad (3.26)$$

The inelastic shear buckling strength between the limits $439\sqrt{k_v/F_y}$ and $502\sqrt{k_v/F_y}$ is,

$$F_s = 290 \frac{\sqrt{F_y k_v}}{h/w} \quad (3.27)$$

The complete curve is illustrated in Figure 3.2.

Figure 3.2: Web Shear Strength-Stiffened Web

3.4 Intermediate and Bearing Stiffener Design

3.4.1 Bearing Stiffeners

Bearing stiffeners are required at all locations where concentrated loads are greater than web capacity for yielding or crippling defined in clause 15.9, or when a web depth-to-thickness ratio is greater than $1100/\sqrt{F_y}$ at the unframed ends of beams.

The bearing stiffeners are composed of a pair of plates that meet clause 3 requirements and are designed as columns (with the appropriate web area included) that can only buckle out of the plane of the web. Figure 3.3 illustrates the geometry of the critical area for interior loads and reactions. It is assumed that the concentrated load or reaction is distributed uniformly over the length of the bearing plate. The resistance

Figure 3.3: Concentrated Loads and Reactions

is defined as the lesser value given by the Equations in clause 15.9 for web yielding and web crippling for the appropriate location, interior loads and end reaction,

- Web yielding

- (i) interior loads

$$B_r = 1.1\phi w(N + 5k)F_y \quad (3.28)$$

- (ii) end reaction

$$B_r = 1.1\phi w(N + 2.5k)F_y \quad (3.29)$$

- Web crippling

- (i) interior loads

$$B_r = 300\phi w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{w}{t} \right)^{1.5} \right] \sqrt{\frac{F_y t}{w}} \quad (3.30)$$

- (ii) end reaction

$$B_r = 150\phi w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{w}{t} \right)^{1.5} \right] \sqrt{\frac{F_y t}{w}} \quad (3.31)$$

where,

N = length of the bearing of an applied load or reaction

k = distance from outer face of flange to web toe of the web-to-flange fillet

3.4.2 Intermediate Stiffener Design

Intermediate transverse stiffeners resist the vertical component of the force caused by the tension field action over one panel width (refer to Figure 3.4), which is equal to,

$$F = \frac{\sigma_t h w}{2} \left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}} \right] \quad (3.32)$$

where, σ_t is the maximum tensile stress that can be applied in conjunction with the critical shear buckling stress and is determined from a straight line approximation of von Mises Huber Hencky yield criterion as $(1 - F_s/\tau_y)F_y$.

Figure 3.4: Web Tension Field

The critical shear buckling stress, F_s is determined by Equation 3.15 and the yield shear stress $\tau_y = F_y/\sqrt{3}$. Substitute these into Equation 3.32 and rearranging the equation to be expressed in terms of the stiffener area, A_s , assuming that the stiffeners are class 3 ($F = A_s F_{ys}$), gives Clause 15.7.3,

$$A_s \geq \frac{aw}{2} \left[1 - \frac{a/h}{\sqrt{1 + (a/h)^2}} \right] \left[1 - \frac{310000k_v}{F_y(h/w)^2} \right] \frac{F_y}{F_{ys}} D \quad (3.33)$$

where,

- F_{ys} = yield strength of the stiffeners
- D = stiffener factor
 - = 1.0 stiffeners in pairs
 - = 1.8 angle stiffener on one side only,
 - = 2.4 plate stiffener on one side only

The value $\left[1 - \frac{310,000k_v}{F_y(h/w)^2} \right]$ must be equal to or greater than 0.1.

The shear flow at each intermediate stiffener is given by an empirical expression in clause 15.7.4 as

$$v = 1 \times 10^{-4} h (F_y)^{1.5} \quad (3.34)$$

The moment of inertia of the stiffener (single or pair) about the axis, in the plane of

the web must meet the following requirement

$$I_s \geq \left[\frac{h}{50} \right]^4 \quad (3.35)$$

To develop the horizontal component of the tension field force at the ends of the plate girder the first panel or anchor panel is sized (distance a) such that tension field action is not required to carry the factored shear force, hence a or h (whichever is least) must be less than $1150w/\sqrt{V_f/\phi A_w}$.

The maximum distance between stiffeners is defined in clause 15.7.2,

$$\text{for } h/w \leq 150, \text{ as } a/h \leq 3 \quad (3.36)$$

$$\text{and for } h/w > 150, \text{ as } a/h \leq \frac{67500}{(h/w)^2} \quad (3.37)$$

3.4.3 Design Procedure for Stiffeners

Bearing and intermediate stiffeners are designed for the full height of the web and consist of a pair of plates. They have thicknesses that vary from 5mm through to 12mm, 14mm through to 20mm, or are 22mm, 25mm, 28mm, 30mm and 35mm. The design procedures for each stiffener type follow.

Bearing Stiffener

1. Calculate the resistance of the web for interior loads and reactions and compare the resistance with the corresponding factored loads at each occurrence. If $B_r < P_f$ or R_f continue, otherwise, stiffeners are not required.
2. Calculate the range of stiffener widths
 - width ≥ 10 mm (minimum width requirement)
 - width ≤ 0.5 [flange width of girder - web thickness]
 - width $\leq 200t/\sqrt{F_y}$ (class 3 requirement)

3. Calculate the width b for a stiffener thickness, $t = 6$ mm.
4. Calculate the column cross - sectional properties (A, I) for a section that consists of a pair of stiffeners centrally located on a web strip equal to but not more than 25 times its thickness for interior stiffeners, or a strip equal to but not more than 12 times its thickness for stiffeners located at the end of the girder. The effective column length, Kl , shall be taken as not less than three-fourths of the length of the stiffeners in calculating the ratio Kl/r . The compressive resistance of the equivalent column, C_r , with the cross-section and slenderness defined by Clause 15.6.2, is calculated from Clause 13.3.1.
5. Bearing on contact area. The bearing resistance for the contact area between the stiffeners and the flange is calculated according to Clause 13.10.1(a) as,

$$B_r = 1.50\phi F_y A \quad (3.38)$$

where,

A = the contact area of a pair of bearing stiffeners
 $= 2(\text{half flange width} - \text{fillet})t$, (assuming a 6 mm fillet weld between the web and the flange)

$$\phi = 0.9$$

6. Compare the axial compressive resistance C_r , to the factored concentrated load or reaction. If the resistance is less, then select the next large plate thickness, otherwise the design is satisfactory.

Intermediate Transverse Stiffeners

1. Calculate the maximum factored shear stress and compare it with F_v for unstiffened webs, $k_v = 5.34$, to determine if intermediate stiffeners are required, where, $f_s = V_{f_{max}}/hw$.

2. Establish end panel spacing according to Clause 15.7.1, (elastic behaviour tension field action not required).

- $a < 1150w/\sqrt{V_f/\phi A_w}$ or $a < h$
- round down the end panel distance.

3. Determine intermediate stiffeners

- Calculate the factored shear at distance a (end panel spacing) from the end of the span and the corresponding factored shear stress, f_s .
- Use maximum allowable stiffener spacing defined by Equation 3.36 or 3.37, as a trial stiffener spacing.
- Calculate k_v from Equation 3.24 or 3.25, for the current aspect ratio.
- Calculate ultimate shear stress F_s from Equation 3.23, Equation 3.26, or Equation 3.27.
- If the ultimate shear stress is less than the factored shear stress, then the distance a is reduced by 50mm and the process repeated until sufficient shear resistance is provided. Repeat this series of steps for each intermediate stiffener (where the distance to the critical section is equal to the distance from the end of the span to the location of the last intermediate stiffener) until the entire span has been evaluated. The exact stiffener layout will be determined after the bearing stiffeners have been determined.

4. Design the intermediate stiffener, area, moment of inertia and welds according to Equation 3.33, Equation 3.34 and Equation 3.35.

3.5 Combined Shear and Moment

The program PGIRD checks the combined shear and moment resistance of the plate girder at all critical locations where,

- (i) the factored shear force at the section being considered exceeds 60% of the shear resistance of the section, and
- (ii) any location where the cross-section is reduced, and
- (iii) if tension field action is required to carry the factored shear, $h/w > 502\sqrt{\frac{E}{F_y}}$, according to the shear, moment interaction equation defined in Clause 13.4.1.4,

$$0.727\frac{M_f}{M_r} + 0.455\frac{V_f}{V_r} \leq 1.0 \quad (3.39)$$

where, V_f and M_f are the factored shear force and moment at the location of interest and V_r and M_r are the corresponding resistances.

3.6 Weld Detailing

Fillet welds are used to transmit the shear flow between the web and flange, and between the stiffeners and web. The shear resistance of a weld must be evaluated on the basis of both the resistance of the weld as defined in Clause 13.13.1 and Table 3

(A) as being the lesser of

- (i) the shear resistance of the weld metal,

$$V_r = 0.67\phi_w X_u A_w \quad (3.40)$$

or

- (ii) the shear resistance of the base metal,

$$V_r = 0.67\phi F_y A_m \quad (3.41)$$

where,

V_r = factored shear resistance per mm length

ϕ_w = resistance factor, 0.67

X_u = ultimate strength of the weld metal as given by electrode classification number

A_w = effective throat area of weld per mm length

A_m = area of the fusion face per mm length

ϕ = resistance factor for the base metal, 0.90

F_y = yield strength of the base metal

Web-to-Flange Connection

The shear flow that must be transmitted across the interface between the web and flange is,

$$q = \frac{VQ}{I} \quad (3.42)$$

where,

q = shear flow, kN/mm

V = the maximum factored shear force

Q = the first moment of the flange area about the neutral axis of the cross-section

I = moment of inertia of the cross section

The fillet welds are placed in pairs (far side, near side) and have a minimum length 40 mm or 4 times the weld size, whichever is greater. The maximum clear spacing for welds that are not staggered is defined in Clause 19.1.3(a) as 300 mm or $330t/\sqrt{F_y}$, where t is the thickness of flange plate, and in Clause 19.1.3.(b) for staggered welds as 450 mm or $525t/\sqrt{F_y}$. The maximum clear distance between welds is defined in Clause 15.7.4 and is the lesser of 16 times the web thickness or 4 times the weld length. The minimum size of fillet weld is defined by the thickness of the flange plate and is selected according to CSA W59-M1989. The resistance provided must exceed the shear flow required and is calculated as,

$$q = \frac{2V_r l}{s} \quad (3.43)$$

where,

V_r = shear resistance of the weld defined by the lesser of Equation 3.40 and Equation 3.41

l = length of welds

s = centre-to-centre spacing of welds

Bearing Stiffener-to-Web Connection

The total length of weld required to transfer the concentrated load from the bearing stiffener to the web is,

$$L = \frac{V_f}{V_r}$$

where,

V_f = factored concentrated load

V_r = shear resistance of the weld

If the bearing resistance of the web is included, this can be reduced to,

$$L = \frac{V_f - B_r}{V_r}$$

The minimum weld length, maximum clear distance, and centre-to-centre spacing are the same as that for web-to-flange welds.

Intermediate Stiffener-to-Web Connection

The shear flow required to be transferred along the interface of intermediate stiffeners and the web is defined in Clause 15.7.4 as,

$$v = 1 \times 10^{-4} h F_y^{3/2} N/mm \quad (3.44)$$

The shear resistance results from four intermittent welds of minimum length located on the far and near side of the pair of stiffeners is given by,

$$v_r = \frac{4V_r l}{s} \quad (3.45)$$

where,

l = length of weld, 40mm

s = centre-to-centre spacing of welds

The center-to-centre spacing is calculated by equating Equations 3.44 and 3.45.

3.7 Girder Splice

Girder splices are require in any girder where length or weight limits for transportation, fabrication or erection, restrict the size of the girder to be less than that required. Web and flange splices transmit the internal forces, shear in the web and moment in the flanges, from one member to another.

According to OHBDC 1991, bolted splices should also be proportioned to resist a minimum load equal to 75% of the factored resistance of the smaller of the connected members , based on the behaviour of the members in the overall structure. Complete joint penetration groove welds should be used for welded splices.

Splices are generally bolted, made in the field and positioned at locations of minimum moment. Moment splices consist of single plates equal in area to the flange and transmit the force through a series of bolts acting in single shear. Shear splices consist of two splice plates, one on each side of the web, that extend the entire depth of the girder from flange to flange. PGIRD gives the moments and shears at recommended locations along the girder.

A splice plate connection design algorithm has not been included in PGIRD as it is considered to be outside the scope of this thesis. However, a subroutine has been

written to determine the resistance of eccentrically loaded weld and bolt groups, and could be used in conjunction with the internal forces given by PGIRD to design the connections.

Chapter 4

Interactive Graphic System for Steel Flexural Members

4.1 General

Many researchers, Anumba, C.J., and Watson, A.S. (1992), Rahimian, A. (1992, Chuang, L.C. and Adeli, H. (1993) have been developing interactive graphic programs for the design of structures. He, Y. (1993) developed IDFSM, an interactive-graphic program for the analysis and design of steel flexural members subject to flexure and torsion. IDFSM is limited to single span steel members (with or without overhangs, Gerber system) and to cross-sections that are prismatic.

The graphics user interface of IDFSM was modified to incorporate the design of continuous prismatic and non-prismatic plate girders and renamed IDCFCM, Interactive Design of Continuous Flexural Members.

IDCFM includes the finite element analysis and the plate girder design procedure described in chapters 2 and 3, respectively. The definition of structural geometry, preparation of load data, structural analysis, member design and 3D rendering of

the member are embedded together in the program IDCFCM with seamless automatic communication between each process.

The user is provided with simple comprehensive information at each stage during the development of the model for the problem, comprehensive reports of the analysis and design and figures at each stage that illustrate the corresponding information. The emphasis is placed on providing the user with the tools to consider design alternatives so that sound engineering decisions based on performance requirements, fabrication, erection and costs can be made.

The computer-graphics features are programmed in C within a computer graphics software package called Hoops.

4.2 Implementation of Data Pre and Postprocessors, Analysis and Design Modules

The IDCSM program consists of three independent modules, each of which contain all the subroutines required to perform a specific portion of the processing task associated with input of data (preprocessing), structural analysis, design or output of data (post-processing). The modules include the user interface module which was developed by He (1993) and modified for this thesis as described in this Chapter, a BEAM-DESIGN module which incorporates the analytical programs TORSTEEL and PLFRAM, He(1993), and the design PGIRD module for the analysis and design of plate girders.

4.2.1 The Data Preprocessor

The data preprocessor includes all of the function modules associated with the development of the structural model which include the selection of the structural system, member definition (section and length), definition of boundary conditions and lateral support, connection definitions, and load definition; and modules for editing and translating the input data for the structural analysis module. The preprocessor forms the largest portion of the graphic user interface.

IDCSM graphic interface gives complete on-screen graphic depiction of each input item which is input through a display window by the keyboard. Data files with extension .in and .Data can be retrieved modified and acted on by modules within IDCSM. The data is stored in a general database which is defined for the entire integrated system and shared by all analysis and design modules.

4.2.2 Analysis and Design Processor

The structural analysis and design of the plate girder is executed in the PGIRD module within IDCSM. PGIRD contains the routines for all functions displayed on **Automatic Design, Check Member and Internal Forces & Deflections** of the main menu which includes preliminary sizing, the finite element analysis program for continuous flexural prismatic and non-prismatic members with any number of specified load combinations, member design, and the display of numerical results. Note, the preliminary section sizes and the corresponding geometric properties are computed by algorithms within PGIRD, and displayed. These can be redefined. The execution time is a function of the number of span and load combinations, but generally, only takes a few seconds.

4.2.3 The Data Postprocessor

The data postprocessor displays the numerical and graphical results generated by analysis and design procedures. The numerical output, a summary report and detailed report of the design, include sufficient information so that the structural engineer can verify or modify the design if required. Graphical output includes deflected shape, shear force and bending moment diagrams and a 3D rendition of the plate girder design.

4.3 User Interfaces

The graphics-oriented user interface for the program IDCSM has been designed to be 'user-friendly', to give engineers the capability to manipulate user-defined data, to give input data, and analytical and design data in graphical form or scaled 3D renditions to simplify the review and confirmation procedure. The screen is generally subdivided into a series of windows which drive the process, display results of a given action or allow for data input and include a display window, a structural image display window, an additional information or data input window and view control menu as shown in Figure 4.1. The text-based major menu at the top of the screen is designed for the analysis and design process, and gives access to all subordinate tasks when activated. An illustration of this is shown in Figure 4.2 for the definition of plate girder geometry. Subwindows that prompt for data entry appear automatically, leading the engineer through the process. Data is entered using the keyboard and is finished by pressing the **Return** or **Enter** key. The units of input and output data are shown in the corresponding windows. For most application windows, activating the **Return** button will return the program to the main menu shown in Figure 4.3. The flow of the design process is directed by the user through the selection of a menu item.

Figure 4.1: Window Definition for a Typical Input Screen

Figure 4.2: Definition of Girder Geometry

4.4 User Interface for the Design Process

The design process requires input from the structural engineer to specify the structural system, to define the geometry of the flexural member (prismatic, number of spans, lengths), the support conditions, the applied loads, the design constraints and the type of solution. The main menu screen for data input is illustrated in Figure 4.3. Each button under flexural member description and that for member evaluation and results, calls a series of subordinate interface screens that were designed for that part of the process. These will be described in detail, with examples in the following sections. Design constraints and design options for evaluating the moment resistance require single value input or are toggle switches and are straight forward in their application (requires no further explanation).

Member Evaluation, Design Constraints and Design Options			File: In	Date: 14-Sep-1994
Member Evaluation and Results			Return	Hardcopy name: In1.ps Please wait...
FLEXURAL MEMBER DESCRIPTION			DESIGN CONSTRAINTS	
Structural System	NotOk	Define/Check	Camber for Dead Load: No Change	
Member Length, Section Type & Size	NotOk	Define/Check	Deflection Limit(Live): L/800 Change	
Support(s), Stiffener(s), Cope(s) & Residual Stresses	NotOk	Define/Check	Maximum Member Depth: 1000.0 (mm) Change	
Applied Loads	NotOk	Define/Check	Minimum Member Width: 100.0 (mm) Change	
Internal Forces & Deflections	NotOk	Check/Execute	Automatic Design Check Member	
DESIGN OPTIONS FOR EVALUATING MOMENT RESISTANCE				
CAN/CSA-S16.1-M89	Yes	Ok	Default option (only for non-cantilever beams)	
Modified SSRC Guide(1988)	No	Change	W2 and K values, and load patterns	
Schmitke and Kennedy(1985)	No	Change	Interactive buckling	
Albert, Essi & Kennedy(1991)	No	Change	Finite element analysis	
<div style="border: 1px solid black; width: 100%; height: 100%; text-align: center; font-size: small;">Structural Image</div>				
<div style="border: 1px solid black; padding: 5px;"> DEFAULT OPTIONS ARE HIGHLIGHTED. USE DEFAULT OPTIONS OR MODIFY SELECTED OPTIONS BY PRESSING THE APPROPRIATE BUTTON </div>				

Figure 4.3: Main Menu Screen for Data Input

4.4.1 Structural System

Activate the **Check/Define** button for **Structural System** shown in Figure 4.3, and the structural system definition screen shown in figure 4.4 will be displayed.

Figure 4.4: Structural System Selection Screen: Gerber Girder

Options include Gerber System and Beam to Girder/Column (He, Y.(1993)), and Plate Girder presented in this thesis, stub Girder, OWSJ and Floor system. The latter three are not active. Selecting the **Plate Girder** button displays the **Plate Girder** structural system definition screen shown in Figure 4.5, with varying support conditions for single span and a specified number of spans for continuous plate girders. Only non-composite laterally supported girders are considered.

Girder System	Beam to Girder/Column	Overlays	File: Edit: Help:
	This Girder System	Steel System	Member name: I-beam Please wait...

Girder to girder at both ends

Girder on girder at both ends

Girder to column at both ends

Two Spans	Three Spans
Four Spans	Five Spans

Continuous plate girder (Three spans)

Noncomposite

Laterally unsupported member

Laterally supported Member (using the top flange)

Laterally supported member

Composite

Beam With Concrete Slab

Beam With Steel Deck and Concrete

Figure 4.5: Plate Girder Structural System Definition Screen

The graphic displays for the remaining steps in the design process are built upon the generic system defined here. The Return button returns to the main menu screen and the indicator beside the structural system button will display Ok. These indicators can only be switched by defining the information required in that step. Member evaluation cannot proceed until all the design input steps have been satisfied.

4.4.2 Member Length, Section Type and Size

Activating the check/define button for the Member Length, Section Type and Section displays the corresponding definition screen shown in Figure 4.6 with the generic image. The process of defining the span length(s) for non-prismatic or

prismatic members is activated by selecting the appropriate button. Prompts for span lengths or lengths where the section is non-prismatic are automatic. Examples are shown in Figures 4.7 and 4.8 for each case. Based on the overall span, preliminary estimates of the section size and the corresponding geometric properties are calculated. These are displayed in the lower window as shown in Figure 4.8.

Span(mm)(Prismatic)	Span(mm)(Non-Prismatic)	Return	File: exam	Date: 5-Oct-1994
Section Size (Prismatic)	Section Size (Non-Prismatic)	Return	Hardcopy name: exam0.ps Please wait...

Original Figure	
		Zoom In	Zoom Out
		Move Right	Move Left
		Move Up	Move down

Figure 4.6: Member span and section size definition screen

To revise the size, activate the Change button. The program will prompt for a new value to be entered with a small window as shown in Figure 4.9. Once the value is entered, the geometric properties will be recalculated and displayed on the screen. The size of each section for non-prismatic sections must be defined using

Span(mm)(Non-Prismatic)		Title: Date: 1-25-1994
Section Size (Prismatic)	Section Size (Non-Prismatic)	Handcopy name: hcl.go Please wait...

Original Figure

Zoom In

Zoom Out

Move Right

Move Left

Move Up

Move down

Figure 4.7: Span Definition for Prismatic Members

this modification routine. Generally, plate girders have a constant depth but varying flange sizes. As a result, this step in the process is not as cumbersome as it first appears. The **Return** button returns the main menu screen and the corresponding indicator will display **OK**.

4.4.3 Support(s), Stiffener(s), Cope(s) and Residual Stresses

The window as shown in Figure 4.10 will be displayed by activating the **Support(s), Stiffener(s), Cope(s) & Residual Stresses** button. The residual stress definition button is reserved for advance finite element analysis to determine the lateral torsional buckling strength. These plate girders are assumed to have adequate lateral support so that this mode of failure does not govern, hence the button is inactive.

Span(mm)(Prismatic)	Span(mm)(Non-Prismatic)	Photo	Title: 0
Section Size (Prismatic)	Section Size (Non-Prismatic)	Return	Hardcopy name: inf.go Please wait...

	Py: 300 Mpa	Change	A = 7120 mm ²	Dead Load = 0.56 kN/m	Original Figure	
	b: 200 mm	Change	J = 170 × 10 ⁶ mm ⁴	Out = 1618 × 10 ⁶ mm ⁴	Zoom In	Zoom Out
	t: 10.0 mm	Change	Ix = 440 × 10 ⁶ mm ⁴	Iy = 20 × 10 ⁶ mm ⁴	Move Right	Move Left
	h: 580 mm	Change	Sx = 1570 × 10 ³ mm ³	Sy = 178 × 10 ³ mm ³	Move Up	Move down
	w: 4.5 mm	Change	Zx = 1084 × 10 ³ mm ³	Zy = 287 × 10 ³ mm ³		

Figure 4.8: Span Definition for Non-Prismatic Members

For single span plate girders, the in-plane support may be defined by a bearing length at either end or as in IDFSM, He (1993), where in-plane support definition includes a description of:

- the W or WWF section providing support;
- the orientation of the column (column web parallel or perpendicular to the orientation of the member (screen), if applicable);
- the end conditions at the support (simple or moment resistant connections, if applicable).

Span(mm)(Prismatic)	Span(mm)(Non-Prismatic)	Plate	File: In Date: 1-Oct-1991
Section Size (Prismatic)	Section Size (Non-Prismatic)	Purlin	Hardcopy name: ind.ps Please wait...

To check the corresponding section size and properly, Select the corresponding section

Please type new value : mm

-

	Fy	300 Mpa	Change	A =	6086 mm ²	Dead Load =	0.62 kN/m	Original Figure	
	b:	253 mm	Change	J =	242 × 10 ⁶ mm ⁴	Cw =	2374 × 10 ⁶ mm ⁶	Zoom In	Zoom Out
	c:	11.0 mm	Change	Ix =	520 × 10 ⁶ mm ⁴	Iy =	30 × 10 ⁶ mm ⁴	Move Right	Move Left
	h:	500 mm	Change	Sx =	1856 × 10 ³ mm ³	Sy =	235 × 10 ³ mm ³	Move Up	Move down
	w:	4.5 mm	Change	Zx =	1942 × 10 ³ mm ³	Zy =	355 × 10 ³ mm ³		

Figure 4.9: Span Definition for Non-Prismatic Members

For continuous plate girders, the program gives a default value for bearing length for each in-plane support which can be changed by selecting the corresponding bearing length in the display window shown in Figure 4.10. Lateral supports can be defined by selecting the **Lateral Support(s)** button. The previously defined structure is shown to scale in the structural image display window in Figure 4.11. The prompts for lateral support definition are automatic and specifically tailored to the previously defined structure.

For new structural problems, lateral supports can be added according to the prompt shown in the display. For existing structures the lateral supports are shown and can be modified or simply reviewed (verified).

For both cases, automatic prompts will lead the structural engineer through the process of defining type and position of lateral support as shown in Figures 4.12 and

Lateral Support(s)	Status	File: In	Date: 7-Mar-1994
Required Bearing Length	Cape(s) and End Station(s)	Return	Hardcopy name: in10.ps Please wait...

Bearing Lengths (mm) Change length of bearing Select support number button	<table border="1" style="margin: auto;"> <tr><td>1</td><td>300</td></tr> <tr><td>2</td><td>300</td></tr> <tr><td>3</td><td>300</td></tr> </table>	1	300	2	300	3	300	<table border="1" style="margin: auto;"> <tr><td colspan="2" style="text-align: center;">Original Figure</td></tr> <tr> <td style="text-align: center;">Zoom In</td> <td style="text-align: center;">Zoom Out</td> </tr> <tr> <td style="text-align: center;">Move Right</td> <td style="text-align: center;">Move Left</td> </tr> <tr> <td style="text-align: center;">Move Up</td> <td style="text-align: center;">Move down</td> </tr> </table>	Original Figure		Zoom In	Zoom Out	Move Right	Move Left	Move Up	Move down
1	300															
2	300															
3	300															
Original Figure																
Zoom In	Zoom Out															
Move Right	Move Left															
Move Up	Move down															

Figure 4.10: In-plane support definition screen

4.13. This process is repeated until all the required or specified lateral supports have been added.

Four typical lateral supports, beam to girder web, beam on top of girder flange, top and bottom chord extension of OWSJ's, and top chord of OWSJ's, connected from either side or both sides can be defined, He (1993).

In-Plane Support(s)	End(s) Support(s)	Print	Doc: in
Foundation Dimensions	Cope(s) and End Stiffener(s)	Return	Date: 7/02/1994
			Hardcopy name: in11.ps Please wait...

1

2

3

10000

10000

Add lateral support(s) at spans (y/n)? ☐

Figure 4.11: Lateral support definition screen

In-Plane Support(s)	Lateral Support(s)	Photo	Title: in
End of main member	Cope(s) and End Sillener(s)	Return	Date: 7-Mar-1994
			Hardcopy name: in12.ps Please wait...

10000

10000

Lateral Support Options							
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><td style="padding: 2px;">W-Shape Beam/Joist</td></tr> <tr><td style="padding: 2px;">Beam to girder w. ...</td></tr> <tr><td style="padding: 2px;">Beam on top of girder flange</td></tr> </table>	W-Shape Beam/Joist	Beam to girder w. ...	Beam on top of girder flange	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><td style="padding: 2px;">Open Web Steel Joists</td></tr> <tr><td style="padding: 2px;">Top and bottom chords of OWSJs</td></tr> <tr><td style="padding: 2px;">Top chord of OWSJs</td></tr> </table>	Open Web Steel Joists	Top and bottom chords of OWSJs	Top chord of OWSJs
W-Shape Beam/Joist							
Beam to girder w. ...							
Beam on top of girder flange							
Open Web Steel Joists							
Top and bottom chords of OWSJs							
Top chord of OWSJs							
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><td style="padding: 2px;">From far side of main member</td></tr> <tr><td style="padding: 2px;">From near side of main member</td></tr> <tr><td style="padding: 2px;">From both sides of main member</td></tr> </table>		From far side of main member	From near side of main member	From both sides of main member			
From far side of main member							
From near side of main member							
From both sides of main member							

Figure 4.12: Lateral support definition screen

In-Plane Support(s)	Lateral Support(s)	Page	File: in Date: 1-Nov-1994
Run about direction	Cope(s) and End Stiffener(s)	Return	Hardcopy name: in13.ps Please wait...

Lateral Support Options							
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><th style="text-align: center;">W-Shape Beam/Joint</th></tr> <tr><td style="text-align: center;">Beam to girder web</td></tr> <tr><td style="text-align: center;">Beam on top of girder flange</td></tr> </table>	W-Shape Beam/Joint	Beam to girder web	Beam on top of girder flange	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><th style="text-align: center;">Open Web Steel Joist</th></tr> <tr><td style="text-align: center;">Top and bottom chords of OWSJ's</td></tr> <tr><td style="text-align: center;">Top chord of OWSJ's</td></tr> </table>	Open Web Steel Joist	Top and bottom chords of OWSJ's	Top chord of OWSJ's
W-Shape Beam/Joint							
Beam to girder web							
Beam on top of girder flange							
Open Web Steel Joist							
Top and bottom chords of OWSJ's							
Top chord of OWSJ's							
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><td style="text-align: center;">From far side of main member</td></tr> <tr><td style="text-align: center;">From near side of main member</td></tr> <tr><td style="text-align: center;">From both sides of main member</td></tr> </table>		From far side of main member	From near side of main member	From both sides of main member			
From far side of main member							
From near side of main member							
From both sides of main member							

Figure 4.13: Lateral support definition screen

Copes and bearing stiffeners can be defined by selecting the **Cope(s)** and **Bearing Stiffener(s)** button and entering the appropriate data through the corresponding interface window (modified version of IDFSM) shown in Figure 4.14.

In-Plane Support(s)	Lateral Support(s)	Photo	Title: in
Cope(s) and End Stiffener(s)	Return	Date: 5-Oct-1994	
		Hardcopy name: in14.ps Please wait...	

Cope Location	Top Flange Tip Copes	Bottom Flange Tip Copes	Top Flange Cope	Bottom Flange Cope

Left Support	Not Applicable	Not Applicable	Not Applicable	Not Applicable
	Change	Change	Change	Change
Right Support	Not Applicable	Not Applicable	Not Applicable	Not Applicable
	Change	Change	Change	Change
Cope length along the beam: (Not Applicable)				Change
NOTE: COPING PATTERN AT EACH APPLICABLE BEAM END MUST BE THE SAME				

Left Support: No Bearing Stiffener Added	Change
Right Support: No Bearing Stiffener Added	Change	
Interior Support(s): Bearing Stiffener Added	Change	
Depth x Width = 426 mm x (?) mm (a pair)	Change	
Thickness = 15 mm	Change	

Figure 4.14: Cope(s) and stiffener(s) definition screen

End copes are not applicable for the continuous plate girder and can only be applied to single span plate girders, or to plate girder systems that are defined as **Girder to girder at both ends** or **Girder to column at both ends**.

The bearing stiffeners are required when the web is subjected to concentrated loads (end reactions and interior loads) which are greater than the local web capacity, given in Clause 15.9 of CAN/CSA S16.1. Bearing stiffeners can be defined for each support by selecting the change toggle switch and by specifying the corresponding thickness. The program PGIRD automatically checks to determine if bearing stiffeners are re-

quired anywhere along the length and will design them according to CAN/CSA S16.1. The stiffeners consist of plates arranged in pairs and have a length equal to the web depth. The maximum width is that of the flange and minimum thickness is 10 mm. The **Return** button returns the main menu screen and the corresponding indicator will display OK.

4.4.4 Applied Loads

The window shown in Figure 4.15 will be displayed by activating the **Applied Loads** button in the main menu.

Dead Load	Live Load 1	Live Load 3	Show All Loads	Photo	File: in	Date: 6-Oct-1994
Wind Up Lft	Live Load 2	Live Load 4	Load Factors	Return	Hardcopy name: in15.ps Please wait.	

Figure 4.15: Load definition screen 1

A maximum of six loading cases and four load combinations can be specified. Significant modifications were made to IDFSM to increase the flexibility of load definition

functions, and to improve communication between the load definitions generated in IDCFM and the plate girder analysis design program PGIRD. Additional functions for load definitions were added to meet the input needs for continuous plate girders.

Load Definitions

Select any load case in the menu shown in Figure 4.15 and the program will automatically prompt for concentrated load values for that case at each point where lateral supports were specified as indicated in Figure 4.16. In the case of dead load definition, the program will automatically apply a uniformly distributed dead load for the self weight of the girder (determined from the section geometry) as shown in Figure 4.17. Once this semi-automatic process is complete the following message will appear prompting the structural engineer to define any additional transverse loads.

Add additional transverse/torsional loads(y/n)

Add load(s) at spans(y/n)?

In general, the answer will be yes and any additional type of transverse load can be applied to the girder at any location along the length. The program currently restricts the definitions of torsional loads for plate girders. Each additional transverse load is defined in response to a series of automatic prompts that are illustrated in Figures 4.17 to 4.21 and can be summarized as,

- definition of load type, refer to Figure 4.17
- if the load type is specified as a concentrated load, then the location and magnitude of the load must be entered as shown in Figures 4.18 and 4.19, respectively.
- if the load type is specified as a uniformly distributed load, then the beginning and end locations of the distributed load must be defined followed by the magnitude, kN/m

Figure 4.16: Load definition screen 2

- if the load type is specified as a linearly varying distributed load, then the beginning and end locations of the distributed load must be defined followed by the corresponding magnitudes.

The process is repeated until the prompt messages

Done in spans(y/n)

is responded with a y for yes. Refer to Figure 4.20. Figure 4.21 illustrates the complexity that can be defined for any load case. All load types are shown. Each load case can be reviewed by selecting the corresponding load case button or the **Show All Loads** menu option. The view control buttons located in the lower right hand corner of the screen must be used to scroll through all the illustrations of applied loads on the girder. Loads are defined as being positive downward. The definitions process can also be used to redefine to positions or types of loads being applied.

Dead Load	Live Load 1	Live Load 3	Show All Loads	Phase	Title: in	Date: 7-Nov-1994
Wind Up Lift	Live Load 2	Live Load 4	Load Factors	Return	Hardcopy name: in17.ps Please wait...	

Load Case 1. Dead Load

Please select load types Done

Load Type	
Concentrated Load	Far side torsional loading
Uniformly Distributed Load	Near side torsional loading
Linearly Varying Dist. Load	Nontorsional loading

Figure 4.17: Definition of additional transverse loads

Figure 4.18: Additional Concentrated load location

Figure 4.19: Additional Concentrated Load Magnitude

Figure 4.20: Termination of Addition Load Function

Figure 4.21: Transverse Load Definition Example

Load Combination

Selecting the **Load Factor** button displays the load combination table on the screen shown in Figure 4.22. The load factor for a given load combination must be calculated in accordance with the National Building Code of Canada (1990), and can be entered by selecting the appropriate box. Prompts will guide the structural engineer through the process.

Dead Load	Live Load 1	Live Load 3	Show All Loads	Print	File: in	Date: 6-Oct-1994
Wind Up Lb	Live Load 2	Live Load 4	Load Factor	Return	Hardcopy name: lnd2.ps Please wait.	

LOAD FACTOR

Load Description	Load Combination 1			
Dead Load:	1.00	Please type new load factor for Dead load: -		
Wind upward	0.00			
Live load 1	0.00	0.00	0.00	0.00
Live load 2	0.00	0.00	0.00	0.00
Live load 3	0.00	0.00	0.00	0.00
Live load 4	0.00	0.00	0.00	0.00

Figure 4.22: Load combination and load factor

The **Return** button will return the main menu screen and the corresponding indicator will display **OK**.

**PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT**

PRECISIONSM RESOLUTION TARGETS

4.5 Internal Forces and Deflections

Activating the **Internal Forces & Deflections** button option in main menu executes the analysis module in PGIRD and displays the screen shown in Figures 4.23. The bending moment diagram for each load combination can be reviewed through the view control buttons and scrolling through the images. Maximum and minimum values are indicated on the diagram. The shear force diagrams and the deflected shapes can be reviewed by selecting the corresponding button and using the view control buttons as before. Examples for each are shown in Figures 4.24 and 4.25. The deflections are calculated using service loads.

Figure 4.23: Bending Moment Diagrams

Figure 4.24: Shear Force Diagrams

Figure 4.25: Deflection Shapes

4.6 Design Results

The design module is executed by activating the **Member Evaluation and Results** button. A summary report giving a table of key ratios of factored loads to resistances for moment, shear and bearing, and a response to design constraint checks is displayed along with a 3D rendition of the designed plate girder in Figure 4.26. This allows the structural engineer a quick informative method to review the design without being burdened with an abundance of detail and numbers. A complete detailed report of the analysis and design can be reviewed on screen by selecting the **Detailed Report** button. All calculations are given making the solution transparent. The structural engineer can check any answer by hand calculation and can review as much or as little information as required.

Figure 4.26: Evaluation Results and Structural Image

Chapter 5

Analysis of Eccentrically Loaded Connections for Girder Web Splices

5.1 General

Prior to 1963, elastic theory was used exclusively in analysing and designing eccentrically loaded welded and bolted connections for steel structures. Research over the last 30 years has led to the development of the effective eccentricity method for elastic design of eccentrically loaded bolted connections, Higgins (1964), and ultimate strength methods based on plastic analysis and the load-deformation response of the bolts or the welds. Abolitz (1966) first proposed the ultimate strength design method, in which each fastener is assumed to achieve its full ultimate resistance at the instant of failure. It was concluded that the plastic method was more consistent with the actual behaviour of the connections than Higgin's method. Crawford and Kulak (1971) used the experimental load-deformation response of individual high strength bolts failing in double shear and an analysis based on the rotation of the connection about

an instantaneous centre to predict the ultimate strength of eccentrically loaded bolt groups. Butler, Pal and Kulak (1972) developed a similar analytical method to predict the ultimate capacity for eccentrically loaded weld groups. Miazga and Kennedy (1986) developed functions to give the ultimate strength of fillet welds loaded in shear for any orientation of weld with respect to load direction. Lesik and Kennedy (1988) furthered the development of the load deformation response of fillet welds loaded at any angle of bending and applied it to predicting the ultimate strength of eccentrically loaded fillet weld groups. These methods for predicting ultimate strength of both eccentrically bolted and welded connections are valid for general case where the line of action of the applied load may have any orientation and may not be restricted to being parallel to a centroidal axis of the bolt or weld group. The solution for this nonlinear problem is even more complicated if the fastener or weld group is non-symmetric.

This chapter describes a computer program that has been developed to predict the ultimate strength of arbitrary bolt or weld groups under an eccentric load with any orientation. The prediction of ultimate load-bearing capacity of eccentrically loaded bolted connections is based on the load deformation response of individual fasteners (Crawford and Kulak (1971)), and for eccentrically loaded fillet weld groups on the nonlinear response of fillet welds which is a function of the angle to which is loaded (Miazga and Kennedy (1986) and Lesik and Kennedy (1988)). A trial iterative procedure is used to determine the ultimate load for the bolt groups and Newton's iterative method is used to solve the set of non-linear equilibrium equations for welded groups. Gauss's formula for numerical integration has been adopted to calculate the ultimate load for welds.

The analysis and solution algorithms presented in this Chapter are necessary for the design of girder web splices. This portion of the design process broaden the scope of the thesis significantly. Consequently, the design of the web and flange splices have not been included in this work.

5.2 Eccentrically Loaded Bolted Connections

Eccentrically loaded bolted connections are frequently used in brackets connected to columns that support beams or girders that are framed eccentrically to the center-line of the columns, simple bolted connections for beams, and in web shear splices. Typical eccentrically loaded connections are illustrated in Figure 5.1. The eccentric load capacity is based on the inelastic behaviour of individual fasteners as determined experimentally (includes bending and shear deformation of the bolt and plastic deformation of the plate bearing on the shank of the bolt) and the assumption that the connection remains rigid, and under load rotates about an instantaneous centre, and that the load is in the same plane as the bolt group.

5.2.1 Shear Load-Deformation Response of Individual Fasteners

The shear load-deformation relationship for a single fastener in double shear can be expressed as,

$$R = R_{ult}(1 - e^{-\mu\Delta})^\lambda \quad (5.1)$$

in which,

R = shear for any given deformation

R_{ult} = ultimate shear strength

Δ = total deformation of the connection including shearing, bending, bearing deformation of the fastener and local bearing deformation of the connection plates

μ, λ = regression coefficients; generally $\lambda = 1.0$ and $\mu = 0.906$ for *S.I.* units

e = base of natural logarithms

Figure 5.1: Typical Eccentrically Loaded Connections

5.2.2 Ultimate Shear Strength

The calculation of ultimate shear strength for eccentrically loaded bolt groups is based on the assumptions that:

1. the connection remains rigid and rotates about an instantaneous centre under an eccentric load, and
2. the ultimate strength of the bolt group is reached when the fastener farthest away from the instantaneous centre reaches its ultimate strength which corresponds to its maximum total deformation.

The shear resistance for the remaining fasteners is calculated from Equation 5.1 and

is oriented perpendicular to the radius from the instantaneous centre. The total deformation in each fastener is equal to the ratio of its distance from the instantaneous centre to the fastener farthest away multiplied by the maximum total deformation, possible for an individual fastener.

Figure 5.2 shows an arbitrary group of m fasteners subjected to an eccentric load oriented at an angle ϕ with respect to the horizontal axis. The distance from instantaneous centre to the i th fastener is

$$r_i = \sqrt{x_i^2 + y_i^2} \quad (5.2)$$

where,

x_i = x coordinate of i th fastener

y_i = y coordinate of i th fastener

The total deformation of i th fastener is

$$\Delta_i = \frac{r_i}{r_{max}} \Delta_{max} \quad (5.3)$$

where,

r_{max} = the farthest distance from the instantaneous centre to the fastener

r_n = the distance from the instantaneous centre to the i th fastener

Δ_{max} = the maximum total deformation generally is equal to 5.0 mm and 4.0 mm for ASTM A325 and ASTM A490 bolts, respectively.

Figure 5.2: Eccentrically Loaded Fastener Group

The horizontal component $(R_x)_i$ of the force R_i for the i th fastener is

$$(R_x)_i = \frac{y_i}{r_i} R_i \quad (5.4)$$

The vertical component $(R_y)_i$ of the force R_i for the i th fastener is

$$(R_y)_i = \frac{x_i}{r_i} R_i \quad (5.5)$$

where, R_i is shear force calculated using Equation 5.1. From equilibrium, the equations of statics for eccentrically loaded bolted connections are,

$$\sum F_x = P \cos \phi - \sum_{i=1}^m \left(\frac{y_i}{r_i} R_i \right) = 0 \quad (5.6)$$

$$\sum F_y = P \sin \phi - \sum_{i=1}^m \left(\frac{x_i}{r_i} R_i \right) = 0 \quad (5.7)$$

$$\sum M_z = Pe - \sum_{i=1}^m (r_i \times R_i) = 0 \quad (5.8)$$

where

ϕ = the angle between applied load and x -axis shown in its positive sense

e = the perpendicular distance from the line of action of the applied load to the instantaneous center

If the applied load is perpendicular to the x -axis and the bolt group is doubly symmetric, then Equation 5.6 becomes

$$\sum_{i=1}^m \left(\frac{y_i}{r_i} R_i \right) = 0 \quad (5.9)$$

and it always satisfied. An iterative procedure was developed to determine the ultimate shear load of eccentrically loaded bolted connections. The program module is simply called EB.

5.3 Eccentrically Loaded Fillet Welded Connections

Eccentrically loaded fillet welded connections discussed in this chapter are limited to cases in which the load is in the same plane as the weld group. Figure 5.3 illustrates typical examples for bracket connections, girder splices and standard beam connection where the weld groups are either symmetrical or asymmetrical and the load is parallel to one of the principal axes of the weld group.

The analytical method for predicting the ultimate load of eccentrically loaded welded group includes a discussion of the theoretical approach based on Miazga and Kennedy (1986), and Lesik and Kennedy (1988); and on the numerical method used for calculating the ultimate load.

5.3.1 Analytical Approach

The analytical approach selected for predicting the ultimate strength of eccentrically loaded fillet welded connections is based upon load-deformation characteristics of the fillet weld, which in turn is a function of the angle of load with respect to the weld segment, equilibrium and compatibility. Generally, it is impossible to obtain a closed form solution which involves the integration of the resistance of the weld group. Hence the weld group is divided into a discrete number of finite weld segments and the ultimate strength of the weld group is equal to the sum of the resistances of these segments. The analytical approach is based on the assumptions that:

1. the connection remains rigid and rotates about an instantaneous centre under an eccentric load,

Figure 5.3: Eccentrically Loaded Welded Connections

2. the ultimate strength of the weld group is reached when the ultimate shear strength and deformation of any weld segment is reached, and
3. the ultimate capacity of a fillet weld subjected to a tension-induced shearing force is the same as for a similar weld loaded in compression-induced shear.

The shear resistance for the remaining weld segments is calculated from Equation 5.30 and is oriented perpendicular to the radius from the instantaneous centre. The shear deformation in each segment is equal to the ratio of its distance from the instantaneous centre to the segment multiplied by the maximum deformation of the

controlling segment (first segment to reach ultimate).

In general, the weld group can be symmetrical, single symmetric or asymmetric and the line of action of the applied load can be oriented at any arbitrary angle ϕ with respect to the x-axis as illustrated in Figure 5.4. A trial location of the instantaneous centre of rotation (i.c.) is selected at a distance x_0 from the vertical leg of the weld group and a distance y_0 from a horizontal centroidal axis of the weld group. The coordinate axes x and y are centered at the instantaneous centre as shown in Figure 5.4.

Figure 5.4: Eccentrically Loaded Weld Group

The radial distance from the instantaneous centre to the i th weld segment is

$$r_i = \sqrt{x_i^2 + y_i^2} \quad (5.10)$$

where,

x_i = the distance from the instantaneous centre to the centroid
of the i th weld segment in the x -direction

y_i = the distance from the instantaneous centre to the centroid
of the i th weld segment in the y -direction

For weld segments located along the horizontal portion of a weld group, the angle between the resisting shear force and the horizontal axis of the weld element of loading is given as,

$$\theta_i = \tan^{-1} \frac{x_i}{y_i} \quad (5.11)$$

and for weld segments located along the vertical portion of a weld group, the angle between the resisting force and the vertical axis of the weld element is given as,

$$\theta_i = \tan^{-1} \frac{y_i}{x_0} \quad (5.12)$$

The critical segment which first reaches its ultimate deformation must be determined. The deformation of all other weld segments can then be calculated by Equation 5.13,

$$\Delta_i = r_i \left[\frac{\Delta_{cr}}{r} \right]_{critical} \quad (5.13)$$

where, r is the distance from the instantaneous centre to the critical segment. The maximum deformation of each weld segment Δ_{max} is a function of the loading angle which is discussed in detail in Section 5.3.2. The horizontal and vertical component of the resisting shear force for each weld segment is based on the orientation of the segment. For segments oriented horizontally, the components in the x and y directions are

$$(R_i)_x = R_i \cos \theta_i \quad (5.14)$$

$$(R_i)_y = R_i \sin \theta_i \quad (5.15)$$

and for segments oriented vertically as,

$$(R_i)_x = R_i \sin \theta_i \quad (5.16)$$

$$(R_i)_y = R_i \cos \theta_i \quad (5.17)$$

where,

R_i = resisting shear force of the weld segment

$(R_i)_{x,y}$ = horizontal and vertical components, the x-direction and y-direction, respectively

For any assumed location for the instantaneous centre, the following equilibrium conditions must be satisfied,

$$\sum F_x = 0 \quad (5.18)$$

$$\sum F_y = 0 \quad (5.19)$$

$$\sum M = 0 \quad (5.20)$$

Substituting Equation 5.14 through 5.17 into Equations 5.18 through 5.20, give the equilibrium equations for the general case,

$$P \cos \phi - \sum_{i=1}^n (R_i)_x = 0 \quad (5.21)$$

$$P \sin \phi - \sum_{i=1}^n (R_i)_y = 0 \quad (5.22)$$

$$P(e) - \sum_{i=1}^n (r_i \times R_i) = 0 \quad (5.23)$$

where, e is the perpendicular distance between the line of action of the load and the instantaneous centre. Note, for the specific case in which the line of action of the load is parallel to the y-axis and the weld group is doubly symmetric, Equation 5.21 is always satisfied. In general, Equations 5.21, 5.22 and 5.23 are transcendental equations, and an exact solution can not be achieved. The use of Newton's numerical method to solve these equations is described in Section 5.4.

5.3.2 Analyses of Fillet Welds Loaded in Shear

The maximum deformation, the ultimate strength, failure plane, and the load-deformation relation of the fillet welds are a function of the angle between the applied load and the longitudinal axis of the fillet weld, Miazga and Kennedy (1986).

Maximum Deformation of Fillet Welds

The maximum deformation of a fillet weld loaded in shear is defined as the deformation of the weld subjected to ultimate shear load $R_{u\theta}$. The normalized maximum deformation at the ultimate load $R_{u\theta}$ is,

$$\frac{\Delta_{max}}{d} = 0.209(\theta + 2)^{-0.32} \quad (5.24)$$

where,

θ = the loading angle defined by Equations 5.11 and 5.12

d = fillet weld size

Ultimate Shear Strength

The normalized expression for the ultimate shear load resisted by fillet weld loaded at any angle θ is

$$\frac{R_{u\theta}}{R_{u1}} = 0.5 \sin^{1.5} \theta + 1.0 \quad (5.25)$$

where,

$R_{u\theta}$ = ultimate shear strength of a fillet weld loaded in shear at any angle of loading

R_{u1} = ultimate shear strength of a fillet weld loaded along its longitudinal axis($\theta = 0^\circ$)

Load-Deformation Response of Fillet Welds

A polynomial was selected to approximate the load-deformation response (an empirical curve fit to test data Miazga and Kennedy (1986)). The expression for the load-deformation response of a fillet weld loaded in shear at any angle θ ,

for $\rho > 0$

$$\frac{R_\theta}{R_{u_\theta}} = -13.29\rho + 457.32\rho^{1/2} - 3385.9\rho^{1/3} \quad (5.26)$$

for $\rho \leq 0$

$$\frac{R_\theta}{R_{u_\theta}} = 8.23384\rho \quad (5.27)$$

where ρ is defined as,

$$\rho = \frac{\Delta/d}{\Delta_{max}/d} = \frac{\Delta}{\Delta_{max}} \quad (5.28)$$

In general, Equations. 5.26 and 5.27 can be expressed in terms of $f(\rho)$ as,

$$\frac{R_\theta}{R_{u_\theta}} = f(\rho) \quad (5.29)$$

Substitute Equation 5.25 into Equation 5.29 and rearrange the expression in terms of the shear resistance at any angle,

$$R_\theta = R_{u_\theta} f(\rho) = R_{u_\theta} (0.5 \sin^{1.5} \theta + 1.0) f(\rho) \quad (5.30)$$

5.4 Solution of Nonlinear Equations

5.4.1 Numerical Integration

Unlike bolts, fillet welds are continuous and must be divided into a number of segments to calculate the ultimate shear load of the fillet weld group due to eccentrically applied

loads. The total resistance of fillet weld group is equal to the sum of the resistances of each segment. Two numerical methods, the summation method and Gaussian integration have been investigated in this thesis for calculating the total resisting force.

Summation Method

The summation method is an approximate integration over a finite interval. A simplified procedure was used to divide the weld group into a number of intervals. The centroid of each interval was selected as a reference point for calculating the resistance and deformation of each interval (element). The total resistance of the weld group is equal to sum of the resistances of all the intervals.

Gaussian Integration

Gauss's formula for numerical integration is based on Legendre polynomials and is given by Cook (1989) as,

$$\int_{-1}^1 f(x)dx \approx \sum_{k=1}^n A_k f(x_k) \quad (5.31)$$

where,

A_k = weighting factor, derived from Legendre polynomials

x_k = the point of subdivision of the interval (-1, 1)

A given definite integral, can be written as

$$I = \int_a^b f(x)dx \quad (5.32)$$

where, $f(x)$ is a known function for which the integral can not be easily evaluated nor can it be conveniently expressed in closed form. To evaluate the integral, a natural

coordinate t must be introduced where, x is expressed as a function of the natural coordinate,

$$x = \frac{b-a}{2}t + \frac{b+a}{2} \quad (5.33)$$

where, a, b are the limits of the function $f(x)$ and,

$$dx = \frac{1}{2}(b-a)dt \quad (5.34)$$

Substituting Equations 5.33 and 5.34 into Equation 5.32 gives,

$$I = \int_a^b f(x)dx = \frac{b-a}{2} \int_{-1}^1 \phi(t)dt \quad (5.35)$$

or in term of Equation 5.31, Equation 5.35 can be expressed as,

$$I = \frac{b-a}{2} \sum_{i=1}^n W_i \phi t_i \quad (5.36)$$

where, W_i are weight factors.

For a six point scheme, the weight factors are,

$$\{W\} = \begin{Bmatrix} W_1 \\ W_2 \\ W_3 \\ W_4 \\ W_5 \\ W_6 \end{Bmatrix} = \begin{Bmatrix} 0.17132449237917 \\ 0.36076157304814 \\ 0.46791393457269 \\ 0.46791393457269 \\ 0.36076157304814 \\ 0.17132449237917 \end{Bmatrix}$$

and the corresponding natural co-ordinates for the six Gauss points are,

$$\{t\} = \begin{Bmatrix} t_1 \\ t_2 \\ t_3 \\ t_4 \\ t_5 \\ t_6 \end{Bmatrix} = \begin{Bmatrix} -0.93246954120315 \\ -0.66120938646626 \\ -0.2386191860832 \\ 0.2386191860832 \\ 0.66120938646626 \\ 0.93246954120315 \end{Bmatrix}$$

5.4.2 Newton Iterative Method

The equilibrium equations given by Equations 5.21, 5.22 and 5.23 can be expressed as,

$$\begin{aligned} f_1(x_0, y_0, P) &= P \cos \phi - \sum_{i=1}^n (R_i)_x = 0 \\ f_2(x_0, y_0, P) &= P \sin \phi - \sum_{i=1}^n (R_i)_y = 0 \\ f_3(x_0, y_0, P) &= P(c) - \sum_{i=1}^n (r_i \times R_i) = 0 \end{aligned} \quad (5.37)$$

where,

x_0 and y_0 indicate the location of instantaneous center, refer to Figure 5.4

$$(R_i)_x = R(x_0, y_0)$$

$$(R_i)_y = R(x_0, y_0)$$

This set of equations is nonlinear with respect to the variables x_0 and y_0 . The solution to these equations gives the location of the instantaneous center x_0, y_0 and the ultimate shear load, P . The roots of these equations, hence the solution, can not be obtained directly. Using Taylor series expansions, Equation 5.37 can be solved by Newton Iteration.

Expanding Equations 5.37 as Taylor series about point (x_{0j}, y_{0j}, P_j) , gives,

$$\begin{aligned} f_1 + (f_1)'_x(x_0 - x_{0j}) + (f_1)'_y(y_0 - y_{0j}) + (f_1)'_P(P - P_j) + \dots &= 0 \\ f_2 + (f_2)'_x(x_0 - x_{0j}) + (f_2)'_y(y_0 - y_{0j}) + (f_2)'_P(P - P_j) + \dots &= 0 \\ f_3 + (f_3)'_x(x_0 - x_{0j}) + (f_3)'_y(y_0 - y_{0j}) + (f_3)'_P(P - P_j) + \dots &= 0 \end{aligned} \quad (5.38)$$

where,

$$\begin{aligned}
f_k &= f(x_{0j}, y_{0j}, P_j) \\
(f_k)'_x &= \frac{f_k(x_{0j} + \delta, y_{0j}, P_j) - f_k(x_{0j}, y_{0j}, P_j)}{\delta} \\
(f_k)'_y &= \frac{f_k(x_{0j}, y_{0j} + \delta, P_j) - f_k(x_{0j}, y_{0j}, P_j)}{\delta} \\
(f_k)'_p &= \frac{f_k(x_{0j}, y_{0j}, P_j + \delta) - f_k(x_{0j}, y_{0j}, P_j)}{\delta} \\
k &= 1, 2, 3
\end{aligned} \tag{5.39}$$

If the values for the interval δ are small then the equation given by 5.38 will be linear. The initial guesses for the variables x_0 , y_0 , and P are x_{01} , y_{01} , and P_1 . The Equations 5.38 will be solved by using Gaussian Elimination. The results from the solution of Equations 5.38 by Gaussian Elimination, $x_0 - x_{01}$, $y_0 - y_{01}$, and $P - P_1$, are taken as increments to the initial guesses. The procedure is repeated until convergence criteria are satisfied,

$$\begin{aligned}
|x_{0j+1} - x_{0j}| &\leq \lambda_1 \\
|y_{0j+1} - y_{0j}| &\leq \lambda_1 \\
|P_{j+1} - P_j| &\leq \lambda_1
\end{aligned} \tag{5.40}$$

and

$$\begin{aligned}
f_k(x_{0j+1}, y_{0j+1}, P_{j+1}) &\leq \lambda_2 \\
k &= 1, 2, 3
\end{aligned} \tag{5.41}$$

where, λ_1 , λ_2 are tolerance values.

The interval δ , the tolerance limits λ_1 , λ_2 have been set to 0.01, 0.00001, 0.0001, respectively. These were determined by trial and error and proved to give good solutions.

After the j th iteration, the final location of the instantaneous center (x_0 , y_0) and the ultimate shear load P for the weld group can be calculated

$$x_0 = x_{0j+1} = x_{0j} + (x_{0j-1} - x_{0j-2}) \tag{5.42}$$

$$y_0 = y_{0j+1} = y_{0j} + (y_{0j-1} - y_{0j-2}) \quad (5.43)$$

$$P = P_{j+1} = P_j + (P_{j-1} - P_{j-2}) \quad (5.44)$$

5.4.3 Guidelines for Determining the Location of the Instantaneous Centre

A trial location for the instantaneous centre must be entered to start the iteration process to determine the true location of the instantaneous centre. The location will always lie on the opposite side of the load about the shear centre. Generally, small offset distances of 20 mm to 50 mm from the shear centre will lead to solutions that will converge. For non-symmetric weld groups, a trial location can be determined by analyzing the weld group for the rectangular components of the load to determine x_0 for P_y and y_0 for P_x , where P_x and P_y are the x and y components of the eccentric load P.

5.5 Numerical Examples

Based on the analytical procedures described, a computer program was developed to calculate the shear resistance of either an eccentrically loaded bolt or weld group. The solution is valid for any arrangement of bolts or weld groups and for loads with any line of action (not restricted to being perpendicular to the x-axis). To verify the correct implementation of the analytical procedure described and to determine the degree of accuracy that can be obtained several hundred examples were evaluated and compared with known solutions and solutions evaluated by hand calculations. Three of these are illustrated here.

5.5.1 Example 1, Summation Method vs Gaussian Integration Method

This example compares the results obtained by the summation method and the Gaussian integration method for a C shaped fillet weld group with an eccentric load shown in Figure 5.5. The ultimate shear load is listed for the summation method and interval lengths varying from 0.02 mm to 0.001 mm and for Gaussian Integration in Table 5.1. The value for the ultimate shear load P , obtained by six point Gaussian integration for each leg of the weld is equal to that obtained using the summation method with each leg of the weld divided into 100000 elements. The Gaussian integration is more computationally efficient.

Figure 5.5: Example 1

Ultimate Shear Resistance (kN)				
Summation Method			Gaussian Integration	
Interval Lengths (mm)				
0.02	0.01	0.005	0.001	
211.6	208.93	207.51	206.51	206.5

Table 5.1: Ultimate Shear Load P by Using Different Methods

5.5.2 Example 2, Comparison with Lesik and Kennedy(1988)

Two different weld groups, their shapes and applied loads are defined in Figures. 5.6 and 5.7, and were compared to solutions produced by Lesik and Kennedy (1988) and those listed in the CISC handbook (1991). The predicted shear resistances of the weld groups for different eccentricities are given in Tables 5.2 and 5.3 for the weld groups illustrated in Figures 5.6 and 5.7, respectively.

Note, the CISC values are based on the analysis presented by Butler, Pal and Kulak (1972) and the values from this analysis are based on the shear deformation characteristics given by Lesik and Kennedy (1988). The latter has recently been adopted by the American Welding Bureau. The predicted values from the present work are in excellent agreement with Lesik and Kennedy (1988) and vary less than 10 percent from the CISC values.

Figure 5.6: Non-Symmetric Weld Group

Figure 5.7: Symmetric Weld Group

α	Ultimate Shear Resistance (kN)		
	CISC (1991)	Lesik and Kennedy	Predicted
0.4	266	268.8	262.2
1.0	143	145	141.2
1.6	95	96	94
2.0	78	78	76
2.4	66	66	64
3.0	53	53	52

Table 5.2: Ultimate Shear Resistance, Non-Symmetric Weld Group

5.5.3 Example 3, Load at any orientation

This example is intended to illustrate the implementation of the procedure associated with determining the shear resistance of a weld group subjected to an eccentric load at any orientation. The non-symmetric and a symmetric weld groups shown in Figures 5.8 and 5.9, respectively, are subjected to an eccentric load with a line of action at an angle of $\phi = 59^\circ$ to the horizontal. The ultimate shear resistances are 63.2 kN and 161.6 kN. The corresponding ultimate shear resistance for the same weld groups for $\phi = 90^\circ$ (the usual case) were 100 kN and 181 kN, respectively. All these results were verified by hand calculations.

Figure 5.8: Non-Symmetric Weld, Load @ $\phi = 59^\circ$

α	Ultimate Shear Resistance (kN)		
	CISC(1991)	Lesik and Kennedy	Predicted
0.2	458	546	540
0.3	458	495	486
0.4	455	443	433
0.6	368	355	346
0.8	305	292	284.5
1.6	176	166	161
2.0	144	135	131
3.0	99	92	89

Table 5.3: Ultimate Shear Resistance, Symmetric Weld Group

Figure 5.9: Symmetric Weld, Load @ $\phi = 59^\circ$

5.6 Summary

In this chapter, the analysis for determining the shear resistance of eccentrically loaded bolt and weld group was presented. The analyses are based on the ultimate load method and are valid for any bolt or weld group and for any eccentric load direction and location in the plane of the fastener group. To solve the problem of integration of transcendental function for welds, Gaussian integration was employed. It is an efficient accurate numerical integration method. It provided results that are

comparable to those provided by the summation method using 100000 weld segments per leg of the weld group. A Newton's iterative method was used for solving nonlinear equilibrium equations, to a given convergence criterion and provided accurate results. The iterative procedure is sensitive to the initial values of instantaneous center and may not converge if the values are too different from the actual values. This computer program can be applied to analyse any eccentrically loaded bolted and welded connection in engineering structures.

Chapter 6

Design Examples: Illustration and Verification

Four design examples are presented, to illustrate the functionality and to verify the analysis and design methods in the program IDCSM. The examples include comparisons with the design of,

- a simply supported non-prismatic plate girder, Kulak et al., (1990), and
- a two span continuous non-prismatic plate girder, Salmon et al. (1990), and
- a simply supported prismatic plate girder with concentrated loads, Kennedy(1993), and
- a simply supported non-prismatic plate girder with multiple loads, McGuire (1968).

6.1 Design Example 1: Kulak et al. (1990)

Kulak et al. (1990) describe the design of a 15200 mm long simply supported non-prismatic plate girder which was designed in accordance with CAN/CSA-S16.1-M89. The program IDCSM estimates the preliminary section size for a prismatic girder on specified values for span, steel grade and deflection limitations. For this example this preliminary size is shown in Figure 6.1. Subsequently, the section lengths for each prismatic section are specified along with the ratio of the moment of inertia of one section to another. Section 1 spans 4500 mm from the supports toward midspan and section 2 spans over the middle 6200 mm. the ratio of I values, I_1/I_2 was specified as 0.67, to be the same as that in the Kulak et al. (1990) example. The applied loads and load combinations are specified and are illustrated in the load definition diagram in Figure 6.2. In this case a uniformly distributed factored dead load of 60 kN/m and a concentrated factored live load of 940 kN at midspan are applied to the girder. The girder was modelled with one finite element. The internal forces, the cross-section size and stiffener sizes and locations are given in Fig. 6.3. Comparisons of the maximum factored moments, the corresponding moment resistances and cross-section sizes for both sections of the non-prismatic girder are given in Tables 6.1 and 6.2 respectively.

Span(mm)(Prismatic)	Span(mm)(Non-Prismatic)	Print	File: example1 Handkey name: exam1.1.ps Please wait...
Section Size (Prismatic)	Section Size (Non-Prismatic)	Return	

To check the corresponding section size and property, select the corresponding section

	Fy: 300 Mpa	Change	A = 17800 mm ²	Dead Load = 1.37 kN/m	Original Figure	
	b: 300 mm	Change	J = 1405 = 10 ⁴ mm ⁴	Cx = 34375 = 10 ⁶ mm ³	Zoom In	Zoom Out
	t: 17.8 mm	Change	Ix = 2923 = 10 ⁴ mm ⁴	Iy = 182 = 10 ⁴ mm ⁴	Move Right	Move Left
	h: 600 mm	Change	Sx = 6797 = 10 ⁴ mm ³	Sy = 820 = 10 ⁴ mm ³	Move Up	Move down
	w: 4.5 mm	Change	Zx = 6949 = 10 ⁴ mm ³	Zy = 1364 = 10 ⁴ mm ³		

Figure 6.1: Preliminary Prismatic Section Size

Wind Up LR	Live Load 2	Live Load 3	Live Load 4	Load Pattern	Return	File: example1	Date: 10-Jan-1993
						Hardcopy name: export 3.gn Please wait...	
<div> <div>Structural Image</div> <div>Zoom In</div> <div>Zoom Out</div> <div>Move Right</div> <div>Move Left</div> <div>Move Up</div> <div>Move Down</div> </div>							

Figure 6.2: Definition of Applied Loads

Member Description, Design Constraints and Design Options		File: example1	Date: 10-Jan-1993																				
Member Section: class 3 member		Hardcopy name: export 3.gn	Please wait...																				
EVALUATION RESULTS <table border="1"> <tr> <td>Moment: (MEd) = 0.89</td> <td>OK</td> </tr> <tr> <td>Shear force: VEd = 0.62</td> <td>OK</td> </tr> <tr> <td>Bearing: BRd = 0.83</td> <td>OK</td> </tr> <tr> <td>MaxDeflection: L/1100(Dead, vertical) L/730(Live, vertical)</td> <td>OK</td> </tr> <tr> <td>MaxDepthLimbs: 2000 mm, MinWidthLimbs: 100 mm</td> <td>OK</td> </tr> </table>		Moment: (MEd) = 0.89	OK	Shear force: VEd = 0.62	OK	Bearing: BRd = 0.83	OK	MaxDeflection: L/1100(Dead, vertical) L/730(Live, vertical)	OK	MaxDepthLimbs: 2000 mm, MinWidthLimbs: 100 mm	OK	REPORT <table border="1"> <tr> <td colspan="2">Member class: Auto-Design</td> </tr> <tr> <td colspan="2">Structural Image</td> </tr> <tr> <td>Summary Report</td> <td>Detailed Report</td> </tr> <tr> <td colspan="2">Concrete Reinforcement Solution</td> </tr> <tr> <td colspan="2">User Specified Solution</td> </tr> </table>		Member class: Auto-Design		Structural Image		Summary Report	Detailed Report	Concrete Reinforcement Solution		User Specified Solution	
Moment: (MEd) = 0.89	OK																						
Shear force: VEd = 0.62	OK																						
Bearing: BRd = 0.83	OK																						
MaxDeflection: L/1100(Dead, vertical) L/730(Live, vertical)	OK																						
MaxDepthLimbs: 2000 mm, MinWidthLimbs: 100 mm	OK																						
Member class: Auto-Design																							
Structural Image																							
Summary Report	Detailed Report																						
Concrete Reinforcement Solution																							
User Specified Solution																							
STRUCTURAL SYSTEM INFORMATION: <p>Structural System Type: Noncomposite Plate Girder (Girder on girder at both ends) Non-Prismatic Member Member Length: Main span = 18800.0 mm The number of section in the span is 3 The length of each section</p> <p>Section number: 1, length: 4800 Section number: 2, length: 6200 Section number: 3, length: 4800</p> <p>Member Section: Class 3 Member</p> <table border="1"> <thead> <tr> <th>Section number:</th> <th>Fy(Mpa)</th> <th>b(mm)</th> <th>t(mm)</th> <th>h(mm)</th> <th>w(mm)</th> </tr> </thead> <tbody> <tr> <td>Section number: 1</td> <td>300</td> <td>340</td> <td>22.0</td> <td>1400</td> <td>10.0</td> </tr> <tr> <td>Section number: 2</td> <td>300</td> <td>480</td> <td>25.0</td> <td>1400</td> <td>10.0</td> </tr> </tbody> </table> <p>Support 1, support length: 300 Support 3, support length: 300</p>				Section number:	Fy(Mpa)	b(mm)	t(mm)	h(mm)	w(mm)	Section number: 1	300	340	22.0	1400	10.0	Section number: 2	300	480	25.0	1400	10.0		
Section number:	Fy(Mpa)	b(mm)	t(mm)	h(mm)	w(mm)																		
Section number: 1	300	340	22.0	1400	10.0																		
Section number: 2	300	480	25.0	1400	10.0																		

Figure 6.3: Design Results, Summary Report

Table 6.1: Comparison of maximum Bending Moments and Resistances

	Maximum Moment (kN-m)	Moment Resistance (kN-m)
Section 1		
Kulak, Adams, Gilmor	3559.5	3584
IDCSM	3559.8	3579
Section 2		
Kulak, Adams, Gilmor	5305	5493
IDCSM	5304.4	5389

Table 6.2: Comparison of Cross-Section Sizes

	b (mm)	t (mm)	h (mm)	w (mm)
Section 1				
Kulak, Adams, Gilmor	300	25	1400	10
IDCSM	340	22	1400	10
Section 2				
Kulak, Adams, Gilmor	500	25	1400	10
IDCSM	490	25	1400	10

The factored moment and resistances given in Table 6.1 are essentially the same with small variances in the resistances resulting from the selection of slightly different flange dimensions. Refer to table 6.2. A comparison of the remaining portion of the design is summarized by the data shown on drawings of each plate girder design in Figure 6.4 and 6.5. Illustrated are stiffener sizes and locations(bearing and intermediate stiffeners), weld sizes, and flange and web dimensions. The variations shown are the result of designer preferences in sizes. The corresponding resistances are satisfactory

6.2 Design Example 2: Salmon and Johnson (1990)

A design example for a two-span continuous welded non-prismatic plate girder subjected to a superimposed uniformly distributed dead load of 11.68 kN/m, concentrated dead loads of 66.74 kN, a girder weight of 4.38 kN/m as illustrated in Figure 6.6, a uniformly distributed live load of 46.72kN/m and concentrated live loads of 267 kN as illustrated in Figure 6.7 is presented in Salmon and Johnson (1990). Lateral support is provided at each support and every 7620 mm between supports. The girder has a constant web depth throughout. The flange plate dimensions are constant in the negative and positive moment regions and are sized to provide sufficient resistance for the maximum moment in that region.

This continuous two span girder was modeled with two finite elements, one span is one element for . Three load combinations considered for the design are,

- Load combination 1: dead load plus live load case 1 and 2 (live loads applied on both span).
- Load combination 2: dead load plus live load case 1 (live loads applied on the left span, only)
- Load combination 3: dead load plus live load case 2 (live loads applied on the right span, only)

The shear force diagrams, the bending moment diagrams, and the deflected shapes for each load combination (the latter group includes one additional figure for the specified dead loads) are illustrated in Figures 6.8 and 6.9, and 6.10, 6.11 and 6.12 to 6.14, respectively. Maximum and minimum values are shown.

A comparison of internal forces, maximum shear forces and bending moments, obtained by IDCSM and Salmon and Johnson (1990) are given in Tables 6.3 and 6.4, respectively. Salmon and Johnson calculated the internal forces by analysing the non-

prismatic girder as a prismatic member. The flanges were then sized to match the maximum positive and negative moments yielding a non-prismatic girder. Tables 6.3 and 6.4 give the maximum shear forces and bending moments for the positive and negative moment regions. Agreement exists for the values considering the analysis as a prismatic member. The girder however is not prismatic and must be analyzed accordingly. This leads to variations in the design moments of 4% and 12% for the positive and negative moment regions, with the latter being under estimated for an analysis as a prismatic member. As a result, the designed section will be smaller than required. Salmon and Johnson's design is based on American Institute of Steel Construction LRFD specification, and IDCSM design on CAN/CSA-S16.1-M89. Different grades of steel were used in both cases. This leads to differences in designs that become more difficult to compare. Relative dimensions are similar. Table 6.5 gives the basic cross-section dimensions for both moment regions and the final design is illustrated in Figure 6.15.

Figure 6.6: Definition of Dead Load

Figure 6.7: Definition of Live Load

Figure 6.8: Shear Force Diagram for Load Combinations 1 and 2

Figure 6.9: Shear Force diagram for Load Combination 3

Figure 6.10: Bending Moment Diagram for Load Combination 1

Figure 6.11: Bending Moment Diagram for Load Combination 2 and 3

Figure 6.12: Deflected Shape for the Specified Dead Load

Figure 6.13: Deflected Shapes for Load Combinations 1 and 2

Figure 6.14: Deflected Shape for Load Combination 3

Table 6.3: Comparison of Maximum factored Bending Moments

	Positive, kNm	Negative, kNm
Salmon and Johnson(1990)	10713	15270
IDCSM (Prismatic)	10712	15270
IDCSM (Non-prismatic)	10278	17142

Table 6.4: Comparison of Maximum factored Shear Forces

	Positive, kN	Negative, kN
Salmon and Johnson(1990)	1642	2442
IDCSM (Prismatic)	1642	2442
IDCSM (Non-prismatic)	1606	2503

Table 6.5: Section Sizes for the Non-Prismatic Continuous Beam, IDCSM

	b, mm	t, mm	h, mm	w, mm
Section 1				
(Positive Moment Region)	460	30	2070	16
Section2				
(Negative Moment Region)	690	38	2070	16

Member Description, Design Constraints and Design Options		Photo	Title: example2	Date: 24-Jan-1995
Member Evaluation and Results		Return	Hardcopy name: exam1.14.ps Please wait...	

EVALUATION RESULTS		REPORT	
Moment: (MMr) = 0.98	OK	Member size: Auto-Design	
Shear force: VMr = 0.70	OK	Structural Image	
Bearing: BMr = 0.99	OK	Summary Report	Detailed Report
MaxDeflection: L/3984(Dead,vertical) L/841(Live, vertical)	OK	CANCSA S16.1-1989 Solution	
MaxDepthLimit= 2100 mm, MinWidthLimit= 100 mm	OK	User Specified Solution	

Structural Image

Figure 6.15: Evaluation Results

6.3 Design Example 3: Kennedy (1993)

The simply supported prismatic plate girder to be designed is 26000 mm long and is subjected to a uniformly distributed dead load of 100 kN/m and two concentrated live loads of 600 kN at midspan and 1200 kN at a distance of 19500 mm from the left support, as shown in Figure 6.16. Only one finite element was required for the analysis. The shear force diagram, bending moment diagram and deflected shape for load combination 1 are illustrated in Figures 6.17, 6.18 and 6.19, respectively. Note, the shear force diagram indicates correctly that the concentrated loads are applied over a finite bearing length. Both Kennedy (1993) and IDCSM designed the girder in accordance with CAN/CSA S16.1-M89. The designed girder illustrated in Figure 6.20 was exactly the same, verifying the implementation of the design module within IDCSM.

Figure 6.16: Definition of Applied Loads, Kennedy (1993)

Figure 6.17: Shear Force diagram, Kennedy (1993)

Figure 6.18: Bending Moment Diagram, Kennedy (1993)

Figure 6.19: Deflected Shape, Kennedy (1993)

Figure 6.20: Designed Girder

6.4 Design Example 4: McGuire 1968

Example 4 involves the design of a 14936 mm long simply supported non-prismatic girder that is subjected to a uniformly distributed dead load of 14.60 kN/m and eight concentrated loads of varying magnitude as shown in Figure 6.21. This girder with multiple loads was also modeled successfully with one finite element. The shear force and bending moment diagrams for load combination 1 for the non-prismatic girder are illustrated in Figure 6.22. For this case, single supported span with symmetric geometry and loads, the maximum internal forces will be the same whether or not the analysis accounts for the girder being non-prismatic, as indicated by a comparison of the maximum internal forces shown in table 6.6. This is not the case, however for the maximum deflections. Slightly more flexible end regions of the non-prismatic girder lead to large midspan deflections as shown in Figures 6.23 and 6.24.

Although the designs were conducted according to different standards and with slightly different steel grades, the designed sections from IDCSM are comparable with McGuire's. Refer to Table 6.7. Variations in flange and web dimensions are the result of designer preferences. The designed girder is illustrated in Figure 6.25. Note, McGuire's plate girder is constructed of built-up plates and angles (stiffeners) riveted together. Comparisons of sizes are based on the overall dimensions of the built-up shape.

Figure 6.21: Definition of Applied Loads, McGuire(1968)

Figure 6.22: Shear Force and Bending Moment Diagrams, Load Combination 1

Figure 6.23: Deflected shape from an Analysis of a prismatic Girder

Figure 6.24: Deflected Shape from an Analysis of the Non-prismatic Girder

Figure 6.25: Designed Girder

Table 6.6: Comparison of Maximum Internal Forces

	Moment, kNm	Shear, kN	
		Positive	Negative
McGuire(1968)	5207	992.5	806
IDCSM	5203	993	806

Table 6.7: Comparison of Section Sizes

	b, mm	t, mm	h, mm	w, mm
Section 1				
McGuire(1968)	457.2	19.05	1828.8	11.11
IDCSM	360	20	1480	11
Section 2				
McGuire(1968)	457.2	35	1828	11.11
IDCSM	540	25	1480	11

Chapter 7

Conclusions and Recommendations

7.1 Conclusions

An interactive graphic program IDCSCM, Interactive Design of Continuous Steel Members was developed to analyse, to design, and to evaluate single span or multiple span prismatic or non-prismatic flexural steel members with realistically prescribed boundary conditions subjected to any set of applied loads.

The analysis for continuous steel members used one beam finite element to model each span. Specifically derived for this work, the analyse strategy is computationally efficient and gives accurate results for both prismatic and non-prismatic members with any set of applied loads. The design of the plate girder is in accordance with CAN/ CSA-S16.1-M89 and current fabrication practice in Canada. Each module within IDCSCM has been tested and verified. Complete comparisons have been made with existing designs for prismatic and non-prismatic single and multi-span plate girders. The results are comparable and only vary in the selection of plate sizes or weld lengths which are a result of designer preference.

The graphic displays developed for IDCSM provides a logical sequence of event driven interface windows that simulate the design process and allow unobtrusive data entry (with as much automation as possible); clear scaled realistically images of the design problem as it is developed by the engineer; and analytical and design results in different formats that allow the engineer to verify the design in as much or as little detail as necessary. The analytical and design program modules are written in C language and can be easily linked with any graphic user interface package. The modular format allows modifications to be made readily. Algorithms and the corresponding computer programs were developed to determine the ultimate capacity for eccentrically loaded bolt and weld groups for any orientation of load and for any orthogonal arrangement of bolts and welds. The algorithm for eccentrically loaded weld groups uses Newton's iterative method to solve the set of transcendental nonlinear equations. Comparisons made with existing solutions from the CISC (1990), Lesik and Kennedy (1988) and from solutions generated by the summation method with a large number of elements verified the accuracy of this program module.

7.2 Recommendations

Advancement in structural analysis, design and graphic representation of the design process parallels the advancements in computer technology. Continued development to reflect this is necessary. Additional advances can be made in:

- expanding the capabilities of the program to accommodate composite plate girders, box girders, decks and composite decks.
- integrating this flexural design module into an integrated system for the analysis, design and fabrication of steel structures. An integral part of this is to link this program with connection design, detailing and estimating modules.

- incorporating the splice connection analysis module which will include the eccentrically loaded bolt and weld group ultimate strength analysis module.
- expanding the capabilities of the analysis program to graphic user interface for haunched members and moving loads.

The proposed beam finite element analysis does not consider the nonlinear behaviour nor out-of-plane stability of steel members. Significant advances can be made by introducing a nonlinear finite element model that will enhance the description of behaviour of realistic steel structures. These advanced models will give the capacity of the member.

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